AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

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766.

Vol. XXXIV.—December, 1895.

NOTES ON HIGH MASONRY DAMS.

By JOHN D. VAN BUREN, M. Am. Soc. C. E.

READ SEPTEMBER 18TH, 1895.

WITH DISCUSSION.

No engineering structure requires more scientific care in its design and construction than a high masonry dam. Where the site is a threatening one and lives are at stake, no element of danger against which prudence and skill can provide should be ignored. It is proposed in the following notes to consider some of the dangers and requirements which govern the design of the section of a straight, high masonry dam.

Conditions of Safety.—Five different conditions of safety are to be considered: 1, against overturning; 2, against sliding; 3, against crushing near the toe; 4, against ice and waves; 5, against undermining. After the engineer has fixed upon these conditions of stability or safety, which express his scientific and practical judgment, the design of the section is easily made by the aid of simple mathematical formulas.

Forces.—In addition to the usual forces assumed to act upon the dam, there will be considered a water pressure on the base under full head. This force is usually ignored in designing the section, under the supposition that the masonry can be either made practically water-tight or drained in such a way as to relieve this pressure. It is, nevertheless, always one of the probable dangers to which such a dam is exposed.

Not only does water under a high head percolate with comparative freedom through the best masonry, but shrinkage or settlement cracks or natural fissures give it free paths, to a greater or less extent, into the body of the masonry or foundation—paths always uncertain, both in direction and area. No skill in the construction of large masses of masonry can prevent the existence of such cracks, nor can all dangerous fissures in the foundation be discovered and closed. They may be horizontal or vertical, longitudinal or lateral, limited or extensive. It is, of course, impossible without enormously increasing the section to provide against all dangerous cracks, but by making it heavy enough to withstand a base water pressure, the safety of the structure against such dangers is increased immensely.

In order to provide against or relieve such a pressure without increasing the section, the plan has sometimes been adopted of draining the masonry and foundation. But should the drains be too small for the leakage, the pressure may remain practically unreduced, being reduced only by the head lost in the flow. Again, should the leakage be large, the flow through the drains under a high head may be powerful enough to wear them away, or dislodge the masonry, and thus destroy them and ruin the dam. The violent vibrations caused by the flow of large streams through such conduits under high heads are certainly an element of danger, and such conduits require the very best masonry.

It would seem to be the safer course to omit all such drains, and, by making the masonry and foundation as tight as possible, to keep the leakage divided into small streams, and to make the section large enough to be entirely stable under a full water pressure on the base. The introduction of this force will entirely change the section usually adopted.

That this is not an imaginary or remote danger is proved by the recent disastrous failure of the Bouzey Dam in France, of which the

section was wholly inadequate to furnish the necessary stability, with a water pressure on the base. In 1882, 442 ft. of this dam moved down stream, without tilting, so as to form a bulge concave to the reservoir and having a versed sine of about 1¼ ft., and badly cracking the masonry at the bends. This year, notwithstanding expensive repairs, involving a heavy reinforcement at the toe and a deep cut-off wall at the heel, it was completely destroyed, resulting in the loss of 150 lives. An examination of the section of this dam shows that no provision was made in the design for a water pressure on the base. It is not at all improbable that the equally disastrous failure of the Habra Dam in Algiers, in 1881, resulted from a similar inadequacy in the section.

In the opinion of the author, a water pressure on the base of high masonry dams should always be considered in the design, *i. e.*, the section should be made large enough to provide for a water pressure on the base at all levels.

Foundation or Ground Pressure or Resistance.—The ground pressure on the base is usually assumed to vary in intensity uniformly from heel to toe, being taken as nothing at the heel and twice the average at the toe. Undoubtedly, in large masses of masonry with sloping profiles, this theory is incorrect, and the maximum intensity is nearer the line through the center of gravity. But the theory covers the worst case (and when the masonry may be considered to act as a monolith, it is correct), and it should be followed in the calculations.

The Toe.—Assuming that the maximum base pressure is at the toe, it is evident that the sharp toes provided for high dams in the profiles of Rankine, and in the usual shoe-shaped dams, do not possess sufficient strength ; the masonry at the toe is of comparatively little value for many feet back, as it would crack away from the main body long before the assumed maximum pressure would be reached. The toe should therefore be bold, with a nearly vertical profile.

The Triangular Section.—Nothing practical is gained by considering any other section than the triangle. This is the true shape at all levels for stability against overturning and sliding. The additional masonry required to give proper crest width is too small in amount to materially affect the problem, and the maximum toe pressure can be provided for by altar courses at the base.

A triangular section can be designed so that it will satisfy at once and precisely both the assumed conditions of stability against sliding

and against overturning, and this is readily modified for crest width, ice pressure and toe pressure, without sacrificing economy. It is evident that the most economical section against sliding is a triangle with a toe angle of 90°, and that the best section against overturning is a triangle with an angle of 90° at the heel. Between these two extremes there is a triangle which will satisfy precisely both conditions of stability, which is the required section, subject to modifications for ice and toe pressure.

Design of the Section.

Assuming that the site has been examined and found safe, the problem is to design the section of the dam under the following assumed conditions:

First.—That there shall be a full water pressure on the base at all levels.

Second.—That the ground pressure shall be a uniformly varying force.

Third.—That the coefficient of friction shall be f.

Fourth.—That the pressure at the toe shall not exceed p per square foot.

Fifth.—That there shall be an ice pressure of I per linear foot of dam at the crest.

Formulas.

The Triangle.—Problem: To find a triangular section of a dam in which the base friction with a coefficient of friction f will exactly balance the horizontal forces, and in which the resultant force will pass through the outer end of the middle third of the base.

Let h = height of dam = $1 = c\,r$, Fig. 1.

b = base of dam = $r\ h$, r being a coefficient.

$y = v\,b = r_1\,h$, r_1 " "

$x = v\,a = r_2\,h$, r_2 " "

s = specific gravity of the masonry = $\frac{w}{u}$.

*w = weight of a cubic foot of masonry = 156 lbs.

u = " " " water = 62.5 "

f = coefficient of friction = 0.65.

R = resultant ground pressure on base.

*This may be increased a little by including the amount of water absorbed.

The center of moments is taken at P, the center of pressure of the water on the back.

$$p\,b = \frac{b}{3}.$$

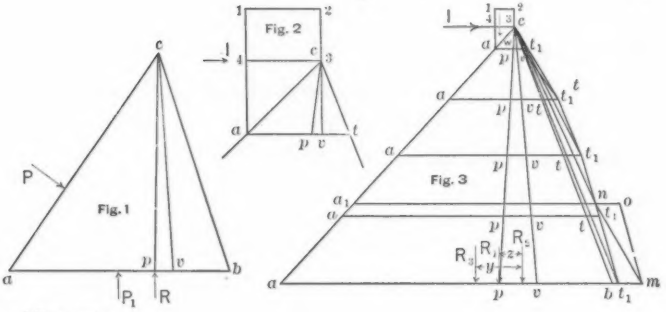
Equations of Equilibrium.

Forces.—Vertical, $\frac{x}{2} + \frac{x+y}{2} \cdot s - (x+y) - R = 0.$

Horizontal, $f\ R = \frac{1}{2}$, or $R = \frac{1}{2f}$.

$$\therefore\ x + y = b = \frac{1 + f\,y}{f\,(s-1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$y = \frac{b\,f\,(s-1) - 1}{f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$



Moments.—

$$\frac{s\,x^2}{6} + \frac{s\,y}{2}\left(\frac{2}{3}x + \frac{y}{3}\right) - (x+y)\left(\frac{x+y}{2} - \frac{x}{3}\right) - \frac{1}{2f}\left(\frac{2}{3}(x+y) - \frac{x}{3}\right) + \frac{1}{6} = 0.$$

Or, as $x + y = b$:

$$s\,b^2 - b^2 - 2b\,y - \frac{b}{f} - \frac{y}{f} + 1 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

combining (2) and (3):

$$b = -\frac{s-2}{2f\,(s-1)} + \sqrt{\frac{1+f^2}{f^2\,(s-1)} + \left(\frac{s-2}{2f\,(s-1)}\right)^2} \quad \ldots\ldots\ldots\ldots (4)$$

$$y = -\frac{s}{2f} + \sqrt{s - 1 + \frac{s^2}{4f^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where b and y are expressed as fractions of $h = 1$.

Example.

Let $s = 2.5$ then $y = 0.357\,h$. If $h = 250$: $y = 89\frac{1}{4}$.
$f = 0.65$ $b = 1.263\,h$. $b = 315.8$.

Modification for Ice Pressure, I, Fig. 2, at Crest.—Add a block of masonry, $123\,a$, at the crest to resist the ice pressure, I, of such dimensions as to give the necessary friction on 34 to resist sliding, and at the same time to pass the resultant of I and the added weight through p, a point in the line of pressure of the simple triangle already determined distant one-third $a\ t$ from t. This block will simply balance the thrust I against sliding and rotation about p. Against sliding from I alone, there will be

$$f\,w\,a\,d = I \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

where a is the crest width of the block 1234; and d its height, 23.

Against rotation about p, there will be

$$\frac{w\,a^2\,d\,(1-2q)}{2} + \frac{(w-u)\,a^2\,h\,(2-3q)}{3} = Ih$$

where q is the fraction $\frac{p\ v}{a\ v}$, (Fig. 2).

The triangle $43a$ replaces the same area of water section and therefore adds only the difference of weights corresponding to $w - u$.

Introducing $h = \frac{a}{r_2}$ and $d = \frac{I}{f\,w\,a}$,

it will be found that

$$a = \sqrt{\frac{3\,I\,(2f - r_2\,(1-2\,q))}{f\,(2-3\,q)\,(w-u)}} \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

Example.

Let $I = 40\,000$ lbs. $r_2 = 1.263 - 0.357 = 0.906$, say 0.91.

$q = 0.07$. $f = 0.65$, say 0.6

$\therefore\ a = 22.3$ ft. and $d = 19.2$ ft.

Areas: $\underline{1234} = 428$ sq. ft.

$\underline{34\,a} = 273$ sq. ft.

Weights: $\underline{1234} = 66\,768$ lbs.

$\underline{34\,a} = \underline{25\,525}$ "

$123\,a \quad = 92\,293$ " $= W$.

The distance of the center of gravity of the added weights, W, from $c = g = 12$ ft.

The values $f = 0.65$ and $s = 2.5$ represent average values. For values of f see Rankine's "Civil Engineering" and the author's paper on "Quay and Other Retaining Walls."*

* See the *Transactions* of the American Society of Civil Engineers, Vol. II, p. 193.

Modification for Ice Pressure below "at," Fig. 2.—Consider h to vary, so that Ih will represent the moment of the ice pressure about p at any level.

Let z = distance which I moves the resultant $R + W = R_1$, toward the toe, Fig. 3. M = moment of W about p at any level. Now let a triangle, ctt_1, be added to the face, with a base $x = tt_1$, in order to equalize the moment Ih, and thus keep p undisturbed.

Then:

$$Ih = R_2 z + M - \frac{r h^2 x}{3}\left(\frac{w}{2} - u\right) \ldots\ldots\ldots\ldots\ldots (9)$$

where R_2 = new resultant $= R_1 + h x\left(\frac{w}{2} - u\right) = R + W +$

$$h x\left(\frac{w}{2} - u\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

and $\frac{rh}{3}$ is, for simplicity, taken for the arm of the added weights, which leads to an inappreciable error in the value of z.

In order that the new resultant, R_2, shall pass through p_1, the extremity of the middle third of the new base, b_1, it is necessary that

$$\frac{2}{3} b_1 = \frac{2}{3} b + z$$

or,

$$b_1 - b = \frac{3}{2} z = x, \text{ the required increase.}$$

Introducing these in (9):

$$Ih = \frac{2}{3} x\left(R + W + x h\left(\frac{w}{2} - u\right)\right) + M - \frac{r h^2 x}{3}\left(\frac{w}{2} - u\right)$$

or

$$x\,(4\,(R + W) - r h^2\,(w - 2u)) + 2\,x^2 h\,(w - 2u) = 6\,(Ih - M)$$

$$\therefore x = -\frac{4\,(R + W) - r h^2 (w - 2u)}{4\,(w - 2u)\,h} + \sqrt{\frac{3\,(Ih - M)}{(w - 2u)\,h} + A^2} \ldots\ldots (11)$$

Here A is the first term of the second member.

Now, by the aid of (11), find x for several levels, and join the outer ends of x for a new profile. The computations are set forth in the following tables:

h.	R.	W.	$4(R + W)$.	$r h^2 (w - 2u)$.	$4(w - 2u)h$.	A.
250	3 005 000	92 300	12 390 000	2 447 062	31 000	320.7
187.5	1 715 630	"	7 232 000	1 376 473	23 250	251.8
125	751 250	"	3 374 000	611 765	15 500	178.2
93.75	422 580	"	2 060 000	344 118	11 625	147.9
62.5	187 813	"	1 120 000	152 941	7 750	124.8
31.25	46 953	"	557 000	38 236	3 875	133.9
24	.....	"	.....	.....	.....	.....

h.	I.	W.	g.	$3(Ih - M)$.	$(w - 2u)h$.	B.
250	40 000	92 300	− 4	31 110 000	7 750	4 014
187.5	"	"	0	22 500 000	5 813	3 871
125	"	"	+ 4	13 890 000	3 875	3 584
93.75	"	"	+ 6	9 589 000	2 906	3 300
62.5	"	"	+ 8	5 285 000	1 938	2 727
31.25	"	"	+ 10	981 000	969	1 012

$$x = -A + \sqrt{B + A^2}$$

g = arm of W, from p.

$h = 250 \quad x = 6.2$
$187.5 \quad = 7.6$
$125 \quad = 9.8$
$93.75 \quad = 10.8$
$62.5 \quad = 10.5$
$31.5 \quad = 3.8$
$24 + \quad = 0$

Weight of added area = 260 000 lbs.
Water on base = 100 000 lbs.

Net weight = 160 000 lbs. = W_1

Observe that R varies as h^2, and the other terms as h^2 or h, making the computations simple.

Maximum Toe Pressure.

Let p = maximum allowable (total) toe pressure.
p_1 = " " " ground pressure.

or, $p_1 = p - u h_x = 2\left(\frac{R + W + W_1}{b_x}\right)$.

where h_x and b_x correspond to the level at which p_1 is reached.

Assume $\frac{b_x}{h_x} = r^1$ to be constant.

But $R = \frac{u h^2_x}{2f}$

$$\therefore h_x = \frac{f r^1 p_1}{2u} + \sqrt{\left(\frac{f r^1 p_1}{2u}\right)^2 + \frac{2f(W + W_1)}{u}} \quad \ldots\ldots\ldots\ldots (12)$$

For the simple triangle $r^1 = r$ and $W + W_1 = 0$, giving

$$h_x = \frac{f r p^1}{u} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)a$$

$= 189$ ft.

Example.

Let $p_1 = 14\,400$. $r^1 = 1.33$; $W = 92\,300$. $W_1 = 160\,000$.

Then $h_x = 168.5$, corresponding to level $a_1 o$ (Fig. 3).

Below the level $a_1\ o$ corresponding to this height, the base must be increased in order to keep the toe pressure within limits.

Increase of Base Below the Point Where p is Reached.—The resultant pressure is now—

$$R_2 = R_1 + W_1 = R + W + W_1 \ldots\ldots\ldots\ldots (13)$$

when W_1 is the net weight of the portion last added to the front profile.

Now add a block of masonry $n\ o\ m\ t_1$ (Fig. 3) at the toe and extending from the point where the pressure p is reached to the base. Let a provision be made for a reverse water pressure at the toe, having a head h_1, and let the assumption be continued that there is a full water pressure all over the base.

Let R_3 = new resultant = $R + W + W_1$ + weight of toe block $n\ o\ m\ t_1$.

y = distance between R_2 and R_3.

q = distance of center of gravity of block $n\ o\ m\ t_1$ from a vertical through the middle of $m\ t_1$.

d_1 = height of added block.

x = required increase of base.

m = distance of point of application of R_2 from t_1, the new toe = $\frac{b_1}{3}$.

$b_1 = b + 6.4$.

Let M_1 = difference between moment of weight W_1, added on face, and moment of water pressure on the increase of base $b\ t_1 = 6.4$ ft. (Fig. 3).

Then the following three equations of condition result:

$$R_3 = R_1 + W_1 + (w d_1 - u h)\, x$$
$$= R_2 + (w d_1 - u h)\, x$$

$$R_3\, y = w d_1\, x \left(m + \frac{x}{2} - q\right) - u\, h\, x \left(m + \frac{x}{2}\right) - \frac{u h_1^{\,3}}{6} + M_1$$

$$^*p_1 = \frac{2\, R_3}{b_1 + x}\left(2 - \frac{3\,(m + x - y)}{b_1 + x}\right) = \frac{2\, R_3}{(b_1 + x)^2}\,(b_1 - x + 3\, y) \ldots (13)$$

Solving for x

$$x = -\frac{p_1\, b_1 + R_2 - 2 b_1\,(w\, d_1 - u\, h) + 3\, w\, d_1\, q}{p_1 + u\, h - w\, d_1}$$
$$+ \sqrt{\frac{(2\, R_2 - p_1\, b_1)\, b_1 - u\, h_1^{\,3} + 6\, M_1}{p_1 + u\, h - w\, d_1} + A^2} \ldots\ldots\ldots (14)$$

where A = first term of second member.

* Where the formula is the usual one for a resultant acting within the middle third.

Example.

$b_1 = 315.8 + 6.4 = \text{say, } 322 \text{ ft.}$

$m = \frac{b_1}{3} = 107.3.$

$p_1 = 14400. \quad R_2 = 3\,097\,300 + 160\,000 = 3\,257\,300$

$M_1 = 260\,000 \times 60 - 100\,000 \times 104 = 5\,200\,000$

Make $h_1 = 0.$

$$q = \frac{81.5 \times 0.337}{2} = 14.5 \text{ ft.}$$

$\therefore x = 30 \text{ ft.}$

$y = -3.3 \text{ ft.}$

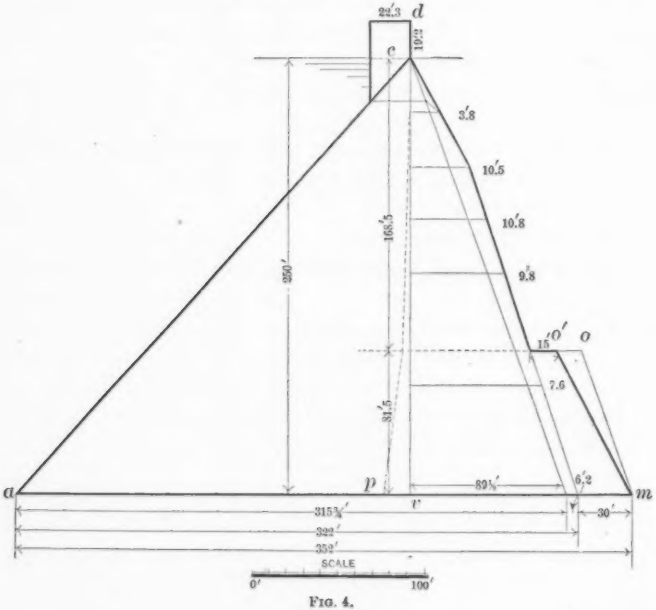


FIG. 4.

This completes the section (see Fig. 4). The block $n\ o\ m\ t_1$ at the toe may, however, without appreciable error, be made triangular, following the line $n\ m$. It will be observed that the inclusion of a back water pressure at the toe will not materially affect the value of x in (14), because its moment for any ordinary depth h_1, which may be taken

as a minimum, will be comparatively very small. But such a pressure will materially increase the stability against sliding below $a_1 n$, and were this alone to be considered, the section might be considerably reduced. By shifting the crest back a small distance on the fixed base $a_1 n$, and thus throwing off some of the overlying water on the back, it is possible to reduce the base slightly at levels above this, and so correct a small part of this excess. But any considerable change of this kind will change the other conditions, and no important improvement can therefore be secured.

The section as now completed satisfies almost exactly the requirements of safety against overturning, ice, toe pressure and sliding. It has only a slight excess of frictional stability between the crest and the base.

Check on the Computations.

The total resultant on the base is—

Simple triangle	= 3 005 000	lbs.
Block 123a at crest	= 92 500	"
Area added to face	= 160 000	"
	3 257 300	"
Less upward effect of toe block $n\,o\,m\,t_1$	= 87 330	"
Total resultant	= 3 169 970	"

Horizontal thrust—

Water on back	= 1 953 125	"
Ice pressure	= 40 000	"
Total thrust	= 1 993 125	"

$$\therefore f = \frac{1\,993\,125}{3\,169\,970} = 0.63$$

But if the toe block, $n\,o\,m\,t_1$, is made triangular, the resultant is reduced 191 710 lbs., half its gross weight, making it 2 979 260 lbs., and

$$f = \frac{1\,993\,125}{2\,979\,260} = 0.66$$

If only half of this triangle is cut off, the toe will have a better shape, the resultant will be 3 074 615 lbs., and

$$f = \frac{1\,993\,125}{3\,074\,615} = 0.65.$$

From (13, second equation) by introducing $x = 30$ there results $y = -3.3$ ft.

$$\therefore p_1 = \frac{2 \times 3\,170\,000}{352}\left(2 - \frac{3\,(107.3 + 30. + 3.3)}{352}\right) = 14\,409 \text{ lbs.},$$

exactly what is required,

The foundation, when not thoroughly good, may be more or less effectively protected against leakage and undermining by an impervious earth filling to any desired height, and such filling, with the section as

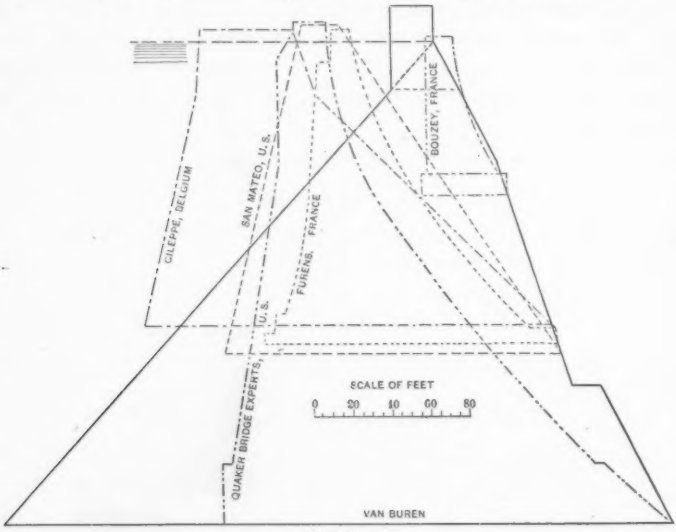


FIG. 5.

designed, will add to the stability in every way. This device, while an additional element of safety, cannot be considered an entirely sure cut-off of water under high pressure. Fig. 5 shows the section drawn to scale, and also the sections of several of the great dams designed in recent years, here and in Europe. Those sections are surely to be commended which, while conforming to theory, provide as far as possible against all dangers. There is no economy without safety, and the engineer who recognizes this truth usually sleeps peacefully on his works.

DISCUSSION.

Mr. Worthen. WILLIAM E. WORTHEN, Past President Am. Soc. C. E.—While the speaker had never built a high masonry dam, he had constructed a number of low ones. In general he had adopted the old form of masonry dam common in New England, which consisted of a mass of common dry rubble masonry, in section a trapezium, with sheet piling on the water side to within 3 to 5 ft. of the crest level, and then sloping up with timber and tight plank covers and often an oak string piece for a crest. But in his dams, with one exception, all wooden constructions are omitted, the sheet piling is replaced by either brick or stone masonry and the cover is of ashlar.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The speaker believed that there was a simple method of avoiding danger from cracks in masonry dams. It consisted in running iron rods up vertically from firm anchorages in the bed rock to the crest of the dam. They would be 15 or 20 ft. apart, and diminish in size from the bottom to the top, as may be indicated by calculation. These rods would be connected by horizontal ties and the entire iron construction would be bedded in the masonry. If such a dam were constructed, no danger would be caused by the presence of cracks, as the iron network would withstand the tensile strains which might occur. That there was a danger of such cracks arising from other causes than the possible expansion and contraction of the masonry was shown by the earthquake which moved up the Atlantic coast several years ago, being most severe in the South. It injured several reservoir dams and allowed the water to escape from them, resulting in the loss of life and property.

Mr. Ridgway. ROBERT RIDGWAY, Jun. Am. Soc. C. E.—In the construction of the Titicus Dam for the Croton Aqueduct Commission of New York, there was danger of settlement cracks where the core wall passed from a rock to an earth foundation. The chief engineer, Alphonse Fteley, M. Am. Soc. C. E., decided to make the wall weakest there in order to be sure where the crack would occur. Accordingly a well was left in the wall at that point, 5 x 8 ft. in horizontal section, so proportioned that the combined thickness of the two side walls was less than that of the normal section of the core wall. This well was carried up about 100 ft. above the foundation. As expected a slight crack showed itself in the side walls of the shaft, but it was so small that it was found unnecessary to fill the well with concrete, so it was filled with earth and the masonry carried up over it to the crest.

Mr. Skinner. F. W. SKINNER, M. Am. Soc. C. E.—The subject of cracks in masonry is discussed in a recent paper by M. Dutoit on the precautions to be taken in the construction of large reservoirs.*

*See the *Annales des Ponts et Chaussées*, June, 1895.

Mr. Skinner. In this article it is stated that it has been found impossible to build large covered masonry reservoirs near Paris, which would not show cracks caused by the variations of temperature. Consequently the last two Parisian reservoirs, at Montmartre and Villejuif, respectively, have been provided with a double bottom, containing a series of intersecting galleries which allow the condition of the masonry to be kept under constant surveillance, and also collect any water which may pass down through the masonry from the basins above. There is a regular system of patching by which the cracks are stopped as soon as discovered. The plaster that is on top of the concrete bottom is removed for about 2 or 2½ ins. each side of the crack through the masonry, which is thoroughly cleaned out by a steel blade and well washed. A strip of board or cardboard about 0.05 in. is inserted in the crack, and Vassy cement 0.4 in. thick is filled in each side of it. Then the strip is withdrawn, leaving a regular opening, and the fresh cement surface is heated and thoroughly dried by charcoal furnaces, after which 1½-in. strips of pure unvulcanized caoutchouc are spread over the crack on the hot cement and soldered with a solution of rubber in benzine. Then the remaining space, about ¾ in. deep, is filled with cement mortar up to the general surface. This last upper layer of cement usually breaks, but the flow of water is stopped by the caoutchouc, while the open space below it allows for the expansions and contractions of the masonry.

A dam is now in course of erection in California, which has some unusual features that deserve mention, although it is not a masonry structure. It is about 130 ft. high above the bottom of the reservoir and 565 ft. long, and is located in a rocky valley. In building it, a trench about 65 ft. wide was dug 30 ft. deep to solid rock, and filled with good rubble masonry. On top of the masonry and running across the whole width of the valley is a web of thin riveted steel plates from No. 0 to No. 3 Birmingham gauge. As each course of plate was calked and completed, it was coated with a special preparation of asphalt. Burlap was then placed over the top edge and each side of the steel, and a coat of asphalt put over it, making two coats on each side of the plate. On each side of the web built up in this manner, there is a 24-in. concrete wall, and on each face of the concrete is a mass of riprap stone sloped 2½ to 1, giving stability to the dam. Dependence is placed upon the steel web, which is built into narrow slots 4 ft. wide and 4 ft. deep in the rock sides of the valley, for water-tight properties, as the filling of large stone on each side offers little resistance to the passage of water.

The largest rock is placed at the lower end of the slope, and nearly 160 000 cu. yds. of it will be contained in the finished structure.

Mr. Clarke. T. C. Clarke, M. Am. Soc. C. E.—The speaker believed it might be interesting to the Society to consider another kind of dam, which is

simple and inexpensive, and can be used even on poor foundations in cases where the smallest discharge of the river is sufficient to always flow over the crest of the dam and keep it wet. It consists of timber crib-work filled with broken stone and its section is shown in Fig. 6. If the bottom is hard the cribs are sunk directly upon it, but if soft, riprap stone of all sizes is dropped into the river, which settles down through the soft material until it will go no farther. In a sand river this sinking can be aided by hydraulic jets. The riprap should be extended for some distance above and below the dam, to form a protection for the river-bed. Upon this foundation after it has settled, cribs 20 to 30 ft. wide, and as long as the section of the dam requires, should be sunk into place, the greater length of the crib parallel to the current. These cribs are not placed close together, but have spaces 5 to 8 ft. wide through which the summer discharge passes while the superstructure is building. Mr. Clarke.

After these cribs have become fixed in place, the superstructure of the dam is built upon them continuously and with no openings in it,

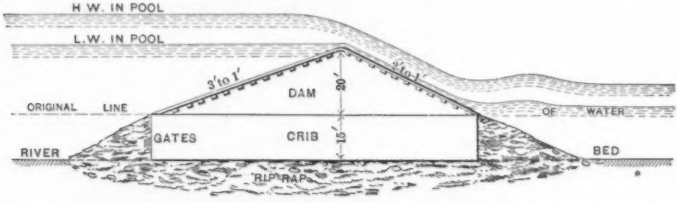


FIG. 6.

clear across the river, and well into the banks. It consists of 12 x 12-in. timbers framed to shape, filled with loose stone and covered on both sides with timbers lying longitudinally up and down stream. They are placed close together and well drift-bolted to the timbers below. The crest of the dam is protected from abrasion by thick plates of rolled iron. After the superstructure is finished, gates of two thicknesses of 12 x 12-in. timber drift-bolted together are placed to close the apertures between the cribs. The water then rises until it flows over the crest of the dam. Such a dam can carry very heavy floods over its crest without injury. Its flat shape on its upper slope makes the water above hold it down, as is the case with dams built by William E. Worthen, Past-President Am. Soc. C. E. The flat slope on the lower side shoots the water up on the top of the water below and prevents its cutting the bed of the river.

A settlement of several feet may take place in some of the cribs without making this kind of dam leak, as it is very flexible. It seemed to the speaker very well adapted for the San Juan River on the proposed Nicaragua Canal, or for such dams as may be required at the foot of

Mr. Clarke. Lake Erie and Lake Ontario to hold up the water to a mean level. A simple and inexpensive sort of material is used that can be had everywhere, and unless the water is allowed to cut around the ends of the dams, floods will not harm them.

Such a dam was built across the Ottawa River by the Department of Public Works of Canada. It is about 1 700 ft. long, and the cribs were sunk in very rapid water from 2 to 23 ft. deep, on a rock bottom. It raises the water 16 ft., and has a lock on one side and a timber slide on the other. It is arched up stream. It has 2½ ft. of water on its crest in summer, and 10 ft. during high floods. No leakage of any magnitude has taken place. Being always submerged, it will last for a very long time.

The speaker designed this dam and many similar ones for the improvement of the navigation of the Ottawa River many years before the structure was actually built. It was very important to know what would be the level of the water at high floods on the crests of the dams, in order to fix the height of the copings of the locks and of the tops of their entrance piers. The following formula was used by him, and also by the engineer of this dam.

Calling the height of water on the crest of the dam H, the length of crest L, the discharge in cubic feet per second Q,

$$\text{then } H = \sqrt[3]{\left(\frac{Q}{3.56\,L}\right)^2}$$

In this case the low-water discharge was 26 000 cu. ft. per second and L was 1 600 ft. Hence $H = \sqrt[3]{\left(\frac{26\,000}{1\,600 \times 3.56}\right)^2} = 2.75$ ft. The actual height observed was 2.50 ft.

The high-water discharge was 190 000 cu. ft. per second, and length of dam 1 760 ft. $H = \sqrt[3]{\left(\frac{190\,000}{1\,760 \times 3.56}\right)^2} = 9.72$ ft. The actual height observed was 10 ft.

This formula is probably as accurate as was the gauging of the river. As this is the test of a very simple weir formula on a very large scale, it seemed to the speaker worthy of record.

Mr. Wegmann. EDWARD WEGMANN, JR., M. Am. Soc. C. E.—The author of the paper recommends a profile for a masonry dam that differs greatly from all types hitherto proposed. This is caused by assuming an upward pressure under the entire base of the dam, equivalent to the head of the water in the reservoir, a condition not assumed by other writers on the subject. Under the requirements imposed, the section is properly proportioned, the formulas used being very simple. If the profile were designed for an extremely unfavorable foundation, the rock being so fissured and porous as to permit the water in the reservoir to exert

Mr. Wegmann.

an upward pressure under a large portion of the base of the dam, the speaker had no criticism to offer except the statement that it is not very probable that an engineer would construct a high masonry dam on such a foundation. The author proposed, however, that the section of a masonry dam should always "be made large enough to provide for a water pressure on the base at all levels," a condition which increases the section very much. The speaker could not see the necessity of taking such an extreme supposition for ordinary cases.

In designing all structures, the engineer has two objects in view, which have opposite tendencies, viz., safety and economy. His skill consists in finding the happy medium which gives sufficient safety with an economical expenditure of money. According to Fig. 5 of the paper, the profile for the proposed Quaker Bridge Dam, designed by a board of experts, would have to be increased about 65% in area to satisfy the author's views. To see whether an upward pressure should always be assumed under the base of a dam, the speaker reviewed briefly what experience had taught about high masonry dams and examined the causes of the failures that have occurred.

So far as he knew, about 55 high masonry dams had been constructed up to the present time. Of this number three had failed, viz., the Puentes, Habra and Bouzey Dams. In each of these cases the failure was due to faulty construction and not to errors in the design. The Puentes Dam was built in Spain during the years 1785–1791. The maximum height of the wall was 164 ft. During the construction of the foundation, a deep pocket of earth was discovered in the center of the valley. Instead of going down to solid rock, the engineers adopted the unfortunate plan of constructing a pile foundation at this place. For 11 years after the reservoir was put into service, the water never attained a greater depth than 84 ft., owing to an insufficiency of rainfall and drainage area. In April, 1802, heavy rains caused the depth of the water to be increased to 154 ft. The result was that the whole pile foundation and soft material in the pocket were forced outward, producing a rupture in the central part of the dam. This accident caused the loss of over 600 lives and the destruction of much property. The failure produced a hole in the central part of the dam, which did not reach, however, up to the crest. The appearance of the central part of the wall after the rupture was that of an arched bridge. Within recent years, this portion of the dam has been repaired and the reservoir has again been put into service.

The second failure mentioned was that of the Habra Dam, constructed in Algiers in 1865–73. A very full account of the destruction of this reservoir wall, with an investigation of the probable causes, has been given by an Italian engineer, Gaetano Crugnola, in the "Ingegneria e Arti di Pareto e Sacheri," Turin, 1882. According to this writer and French engineers who have discussed this case, the failure was not due

Mr. Wegmann. to insufficiency in the area of the profile but to poor material used in the construction and to the strain the wall was subjected to during an unprecedented rain storm. The building material used had to be all obtained near the site of the dam, for reasons of economy. The stone was a very porous sandstone, not uniform in character. The sand used was not very good. The hydraulic lime, made from calcareous stone found on the banks of the Habra River, was not of perfect quality, and contained some quicklime, which probably expanded in the wall and thus assisted in making it porous. Owing to the causes stated, the masonry of the dam was by no means water-tight. When the reservoir was first filled the water passed freely through the wall, which presented the appearance of a great filter. Nevertheless, the dam stood successful for eight years, and did not fail until in December, 1881, when an unusually severe rainstorm occurred, during which 6½ ins. of rain fell in a short time. Crugnola has calculated that in all probability more than 26 000 000 000 galls. of water must have passed over the dam in one night, the sheet of water attaining a height of about 13 ft. above the crest of the dam. This would not have been sufficient to cause the dam to slide on its foundation, but would produce pressures in the masonry near the down-stream face of the wall considerably beyond the limits of safety, without taking into account the strains produced by the impact of the flowing water. The facts stated accounted sufficiently for the failure of the Habra Dam and seemed to the speaker rather to sustain Crugnola's views of the case than the author's statement that the failure was probably due to "inadequacy in the section" adopted for the dam.

The third case of the failure of a reservoir wall was that of the Bouzey Dam, which occurred on April 27th, 1895. An account of the rupture was given in *Le Génie Civil* for May 11th, 1895. The dam was constructed near Epinal, France, and was put into service in 1884. It was founded on porous conglomerate rock, consisting of silicious stones joined together by a very poor cementing material. The foundation trench was carried down to what appeared to be a good bottom, but was not excavated to absolutely solid rock, which was found to lie at a depth of about 20 to 30 ft. in the center of the valley. To prevent leakage under the main dam, a guard wall, 6½ ft. wide, was founded on this compact rock and built up on the up-stream side of the dam. As the maximum depth of the water in the reservoir was to be about 75 ft., this guard wall could hardly be expected to prevent water from percolating under the base of the main dam, causing thus an upward pressure which might destroy the wall's stability against sliding. When the reservoir was being filled for the first time and the water had reached within 11 ft. of the top of the dam, the central portion of the wall slid or bulged forward so as to form almost a regular curve having a chord of about 394 ft., and a versed sine of about 1 ft. There

Mr. Wegmann.

was no settling nor overturning, but simply a sliding or bulging forward. This deformation of the wall produced numerous fissures in the foundation, which discharged a considerable quantity of water, and caused also four vertical fissures in the front face of the wall. Before the reservoir had been filled, two similar vertical fissures had been produced in the face of the dam by changes in the temperature. Water flowed through these fissures during cold weather, but ceased almost entirely in warm weather, owing to the expansion of the masonry. Some attempts were made to close the fissures by means of tarred, wooden wedges, but without success. In 1888–89 the wall was strengthened by building a kind of flying buttress in front of it, which made sliding in the foundation impossible. The upper portions of the wall were not reinforced, however. In 1890 the water in the reservoir was allowed to rise to its highest level without any change being noticed in the wall. The recent accident occurred very suddenly without any warning. The foundation masonry was not displaced, but a breach about 560 ft. long and about 33 ft. deep below the highest water level, was made in the central portion of the dam. In the center of the valley this breach was cut down to the bottom of the reservoir. The portion of the wall which was ruptured contained four of the vertical fissures that have been mentioned. It seemed astonishing that a dam that had given such signs of weakness had been allowed to stand.

The failure of the Bouzey Dam is the only case where an upward pressure under the base of the wall is likely to have been the cause of the disaster. In all probability this pressure would not have occurred to any extent if the whole dam had been founded on the solid rock upon which the guard wall was built.

Very few of the dams that have been constructed would be standing if they were subjected to such an upward pressure. As they have remained stable, some of them for many years, it may be safely concluded that they are not subjected to such an upward pressure to any extent. In this connection the Almanza Dam might be mentioned, which has stood for over three centuries.

The speaker could not agree with the author in his statement of the impossibility of making a dam practically water-tight and of preventing cracks and fissures in the masonry. The Aqueduct Commissioners of the city of New York have constructed, within recent years, two masonry dams in the Croton water-sheds that may be taken as fine examples of what can be accomplished by proper care in the construction. The first of these, the Sodom Dam (98 ft. high above the foundation), has been fully described in a paper read before the Society in March, 1893, by Walter McCulloh, M. Am. Soc. C. E.* The second, the Titicus Dam, has a maximum height of 135 ft. above the foundation. In both cases, the front face of the wall appears per-

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVIII, p. 185.

Mr. Wegmann. fectly dry in clear weather, there being no signs of leakage. In humid weather some damp spots appear on the walls. Water which may force its way through the pores of the mortar and stone is in a capillary state, and not likely to exert the hydrostatic pressure due to head of water in the reservoir. With the exception of a slight hair crack in the overfall of the Titicus Dam, there is absolutely no crack to be found in the two cases mentioned. The expansion of the masonry is probably taken up by the elasticity of the mortar. As regards settling, the slowness with which a large masonry dam is carried up prevents cracks from this cause.

In the construction of the Sodom and Titicus Dams some fissures had been encountered in the foundation rock, and had been closed by masonry. At these points it was very probable that an upward pressure existed, but the area of such points was a very small percentage of the total area of the base of the wall.

The speaker agreed fully with the author as regards the inadvisability of constructing drains in the foundation. Such channels cause additional leakage, and ought to be avoided. With reference to the danger that might exist if the front face of the dam formed a sharp toe, there was a difference of opinion among engineers. Prof. Rankine stated that the pressures near the front face of a dam were tangent to the curve of the face. If such were the case, the pressures might pass along the front face somewhat as the strains in a masonry arch.

The speaker said, in conclusion, that while the assumption of an upward pressure on the base of a dam diminished the stability of the structure against sliding, the hypothesis being in the direction of safety, it diminished, on the other hand, the calculated pressures in the wall, making the masonry appear less strained than it might actually be. The great danger for a high masonry dam was not a possibility of the wall sliding on its foundation, but the great pressures to which the masonry might be subjected near the faces of the dam. For this reason the speaker recommended that an upward pressure under the base of a dam be never considered in calculating the pressures in the masonry, even if it be assumed, in extremely unfavorable cases, in investigating the stability of the wall against sliding.

CORRESPONDENCE.

Mr. Allen. KENNETH ALLEN, M. Am. Soc. C. E.—So far as failure from sliding or overturning is concerned, it seems that the author's proposed section would be sufficient for a well built dam of dry masonry. It appears to the writer, however, that the bond and comparative impermeability imparted by the proper use of mortar modifies the conditions radically.

Mr. Allen. It can hardly be denied that with such masonry as is usually found in well built dams the adhesion of mortar to stone results, to a great extent, in the formation of a monolithic structure, in which the coefficient of friction may generally be disregarded. There are no unbroken horizontal joints of large area, and those which exist are at least nearly full of hard, gritty mortar, clinging more or less to the adjacent stones. Likewise, the assumption of an upward water pressure over the entire bed or base of the dam equal to that of the water at a corresponding depth seems to the writer unjustifiable. Admitting that a certain amount of percolation obtains with an undiminished static head, yet the extreme proportion of such areas of percolation to the entire horizontal section of the dam must be very small in cement masonry. So that, while it is safe to assume a certain amount of upward pressure, as, indeed, will probably obtain, yet this can never act on more than a small fraction of the bed, in the writer's opinion. He is aware that the position held by the author on this point is endorsed by eminent authority, and would not be misunderstood as advocating in any way a section for an important dam based upon any doubtful point without giving the stability of the dam the benefit of the doubt. But as to the influence of cement mortar in increasing the frictional stability and decreasing the total upward pressure, both to an enormous extent, he does not think there is a question. With regard to the calculation of the resultant pressure, he would suggest assuming a certain proportion, say 10, 20 or 30%, of the upward pressure acting on the entire section, depending on the character of the masonry employed, and, in the case of the base, of the bed-rock upon which it is built.

Mr. Duane. JAMES DUANE, M. Am. Soc. C. E.—The area of the cross-section evolved by the author so greatly exceeds that adopted in the best modern practice that it would seem that the premises on which his theory is founded must be in error. This belief is strengthened when it is recalled that in all cases where the materials and workmanship have been good, the generally accepted theory has given ample dimensions. The writer believes that this discrepancy is readily accounted for by the remarkable assumption "that there shall be a full water pressure on the base at all levels." In the few high masonry dams that have come under the writer's notice, all have shown more or less sweating or percolation on the face.

In one or two cases the leakage attained the magnitude of streams at several points, and while this was an eyesore it caused no apprehension as to the safety of the structures. In the worst case where the leakage could be examined at several different points, viz., at the face of the dam, at the interior of a well in the center of the dam, and in the gate-chamber through which the outlet pipes passed, there was no evidence that the upward pressure at any point exceeded a few pounds. This seemed to be independent of the head of water on the particular

Mr. Duane. joint examined, and of the thickness of the intervening masonry between the exposed surface and the water in the reservoir. It may be added that the area acted on by even this slight pressure at any level was insignificant as compared with the area of the entire base at the same level.

Passing now to the generally accepted theory that the base pressure varies in intensity uniformly from heel to toe, being taken as nothing at the heel and twice the average at the toe, the author admits that this assumption is erroneous, and also pleads in extenuation that this erroneous theory covers the worst cases. The writer would ask if the profession generally considers this excuse as a sufficient palliative of the perpetuation of an admitted error. Should not an effort be made to bring this theory into more perfect harmony with the actualities of the case?

The writer believes that while the pressure may be nothing at the heel, that in all normal cases it usually is a definite and considerable amount; that the maximum pressure is at or near the resultant of the various forces acting on the structure; that the pressure at the toe is between that at the heel and the maximum; that the plotted line of pressures would ordinarily be a curve, and that only by the merest accident a straight line, as in the present theory. In dams of insufficient cross-section there may be a negative pressure or tension at the heel, the stability of the structure then being dependent on the excellence of the mortar employed and the time allowed it to set before a strain is put upon it. The writer thinks that it is to the superior quality of the materials employed, and not to any arch action, that such structures as the Bear Valley Dam owe their stability.

Mr. Broomall. C. M. BROOMALL, Jun. Am. Soc. C. E.—It is to be regretted that the author does not enter into the mathematical treatment of the arch-shaped dam. In view of the severe test to which the Sweetwater Dam was recently subjected, such reference would be of interest. The cross-section of the Sweetwater Dam is very bold, and the extent to which its strength is dependent upon the arch shape must be very considerable. It has 70 ft. of water against it, with 46 ft. bottom width and 12 ft. top width, and is built convex up stream, with a maximum radius of 213.3 ft. Its total length is 396 ft. The fact that for 40 hours floods poured over the top of this dam from end to end is a very strong argument in favor of the use of the curved type. Another strong point in their favor is that on account of the curved form the effects of contraction and expansion, due to changes of temperature, would seem less likely to do damage. Also in the case of cracks caused by unequal settlement, the danger would seem to be much less than in the straight dams. The writer would like to ask what changes the author would make in his formulas in order to apply them to the curved dam. It would appear that the treatment would have to be entirely different, almost reducing to that of an arch loaded normally.

M. S. PARKER, M. Am. Soc. C. E.—All engineers must agree with the author "that no element of danger against which prudence and skill can provide should be ignored" in the design and construction of high masonry dams. There is always, however, the question of dollars and cents foremost to capitalists who furnish the means for carrying out the engineer's design. The question arises as to what design is most economical in cost while possessing a sufficient factor of safety, the cost having a considerable influence on the design. The design for a high masonry dam, in the writer's opinion, depends almost entirely upon the character of the foundation to be built upon. Mr. Parker.

There is no disputing the fact that an equilateral triangle is the true shape of a section at all levels for stability against overturning and sliding. This, however, is not the only question the engineer has to decide in the design of such structures. The main question is what design will suffice, in point of safety and economy, to fulfil the prescribed conditions. In addition to the four forces usually considered, the fifth force mentioned by the author, namely, the water pressure on the base under full head, has always been considered by the writer in designs for crib and masonry combination dams for all heights. But in the case of a high masonry dam on suitable foundation, he cannot see the necessity of considering this force such an important factor as to warrant such a change in design as shown between the Quaker Bridge section and the triangular section in Fig. 5 of the paper. It is admitted that water will force its way through the best of masonry, as it will through the best of metal under sufficient pressure, without weakening either material, and that shrinkage or settlement cracks give free paths to a greater or less extent into the body of masonry; but masonry must be very badly constructed that will be so open as to admit water into its interior and hold it under sufficient pressure to endanger its stability to an extent requiring the treatment of the problem proposed in the paper. He is of the opinion that the proper section for a dam should be governed entirely by the character of the foundation to receive it. Admitting the stability of the triangular section for high masonry dams, he questions the necessity of it, assuming the foundation to be solid and suitable to receive the structure.

L. J. LE CONTE, M. Am. Soc. C. E.—The uplifting pressure on the base of dams built upon leaky or seamy foundations has been the subject of much study by engineers of late. Much information is being collected bearing upon the questions involved, but it is of a very contradictory character. In extreme cases, such as the author supposes, 250 ft. in height, the additional amount of masonry required to meet the upward pressure on the entire base is something enormous, over 60%, and the necessity of such an increase is a matter of paramount importance. Mr. Le Conte.

Mr. Le Conte. A study of existing dams and the nature of the ground upon which they are constructed will hardly justify an agreement with the author in his broad statements. There are many large earthen dams in California built on leaky ground which have been in constant use for 25 years or more, notably the old Upper Crystal Springs Dam, near San Francisco. The ground at the site was full of water-bearing gravel beds; the puddle trench was carried down 98 ft. below the bed of the valley, and failed to meet a good impervious stratum. Accordingly the proposed height was reduced to 50 ft. above the bed of the valley, and the dam was built. When the lake was filled springs appeared just below the dam, and continued to flow many years. The dam is 30 ft. wide on top and has a back slope of 3½ to 1 and a face slope of 3 to 1; the water is 45 ft. deep.

Likewise the San Leandro earthen dam at Oakland, Cal., now 120 ft. high above the bed of the valley, rests upon leaky ground, and one or more large springs were struck in sinking the puddle trench some 40 ft. or more. This dam, 20 years old, has always leaked more or less around both ends, through the hillsides, as well as up from the bed below.

The dam is 38 ft. wide on top, and has a back slope of 3 to 1 and a face slope of 4 to 1; the water stands 110 ft. deep.

If the amount of upward pressure on the base of these dams was anywhere near that due to a full hydrostatic head in the reservoirs, it is clear that these dams could not possibly exist. The writer fails to see how such a pressure could exist in any case unless there is an impervious barrier located under the down-stream toe of the dam. In natural ground there is every reason to believe that the amount of ground leakage existing at any dam site will certainly continue immediately below the site. Consequently any leak starting at the up-stream toe of the dam will progress down stream with a diminishing pressure to some natural outfall below the dam. Taken as a whole, then, one would suppose that the upward pressure would not much exceed that due to half the depth of water in the reservoir. Again, it does not seem rational to suppose that this pressure is continuously distributed over the entire base, but only at and near the location of fissures in the bed-rock or foundation. The writer certainly believes in piping all the bad springs found in the foundation and taking them to the lower toe of the dam where they may discharge freely into the creek bed.

In designing large masonry dams another matter must be considered, namely, what may be called the middle third fad. In designing dams of moderate height, this law is harmless and near enough for practical purposes, but when it comes to designing a great dam, 250 ft. in height, then the middle third fad becomes a veritable monstrosity. It leads to an enormous waste of masonry and all to no purpose. Just what law to use as a substitute, the writer is not pre-

pared to say, since the whole subject is fraught with speculation as yet; but it is safe to say that if the oblique resultant (empty or full) comes within the middle half of the base, the actual resultant will be, in point of fact, much inside the middle third. All practical and theoretical knowledge points this way. In short the ordinary straight line formula for distribution of pressure on the base of the dam is of no value whatever when it comes to be applied to the designing of large masonry dams. The toe pressure adopted by the author, 7.2 tons per square foot, is too low entirely; good masonry will stand 10 to 12 tons easily. Mr. Le Conte.

H. F. DUNHAM, M. Am. Soc. C. E.—The author seems to have quite overlooked the advantage derived from the tensile strength of the material used in the construction of masonry dams. It is true that water will percolate through masonry. Imagine a structure like the Quaker Bridge Dam carved out of a single block of porous sandstone; water would find its way through in considerable quantity, but with the water pressure per square inch much less than the tensile strength per square inch of the sandstone, there would be no danger of overthrow. Imagine, now, a horizontal seam dividing the structure at a considerable depth into two parts; the stability would be greatly diminished. But such a seam or crack, not impossible in nature where the scale of operations is proportionately in excess of anything man tries to do, should not be anticipated in any first class masonry construction. It is the function of good work to guard completely against such conditions. It is not evident that settlement or fissure cracks can be of the implied extent. Any one who has had occasion to demolish with drills, bars and dynamite any well built piece of masonry will understand that the "path" area is limited to a very small percentage of the whole area. Mr. Dunham.

A structure like the one referred to might be supposed to rest upon material that had no tensile strength and was quite pervious to water, but no one could fairly be supposed to build upon such a foundation, and it is not usual to generalize for exceptional localities. The foundation and the structure should be properly related, made as nearly as possible like one continuous mass, and the whole treated in each instance so as to insure safety and to avoid needless expense.

JOHN D. VAN BUREN, M. Am. Soc. C. E.—In defining his position, Mr. Wegmann admits that if the assumptions are correct, the proposed section is properly designed. This leaves only one point at issue, the assumption of a water pressure on the base. On this point the late James B. Francis, Past-President Am. Soc. C. E., in his paper on high dams, says: Mr. Van Buren.

"On the base of a wall founded on a rock, having numerous seams, containing water under a head, the flow from which is not carried off by drains, an upward pressure may be transmitted through the mortar to the entire base."*

*See the *Transactions* of the American Society of Civil Engineers, Vol. XIX, p. 169.

Mr. Van Buren. In discussing Mr. Francis' paper, Joseph P. Frizell, M. Am. Soc. C. E., says:

"No dam, as ordinarily constructed, can be wholly free from action of this kind."*

F. Collingwood, M. Am. Soc. C. E., in the same discussion, says:

"The experiments on transmission of pressures through the mass seem to be conclusive, and certainly point to the necessity of taking account of the lifting force of the water."†

The cases of failure cited prove that water under high head is liable to get under dams built by skilful engineers, making the best use of available material, and with appalling results. It seems like begging the question to contend that the failures were due to poor

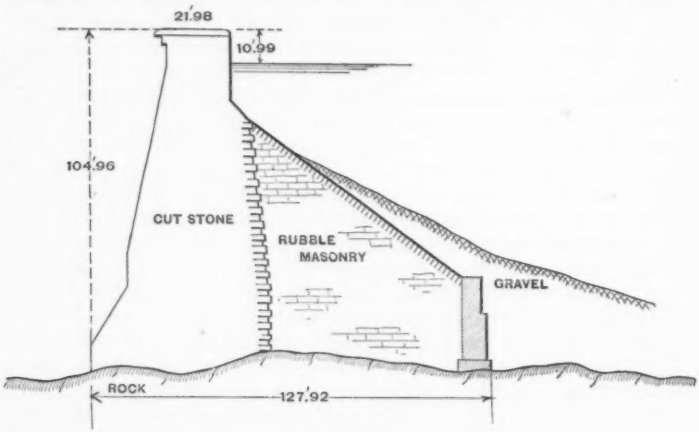


FIG. 7.

material, or that if the rock foundations had been deeper there would have been no failure. Had these dams had such sections as are here advocated, there could have been no such failures.

Mr. Wegmann cites the Almanza Dam, which has stood for three centuries, with a section far below the standard here advocated. But he omits to state that this is a curved dam with a very short radius, 86 ft. How could it be forced out? The (Spanish) Lozoya Dam (Fig. 7), an excellent example of engineering described in Mr. Wegmann's able work on "High Masonry Dams," has a section exceeding the one proposed, which is probably based upon the same theories. It was built by the royal engineers, who evidently appreciated all the dangers which might threaten its safety. Again, the Gileppe Dam has a section greatly exceeding the proposed standard, and, even if it were

* The same, p. 184.

† The same, p. 189.

raised 50 ft., it would still exceed it. M. Bidaut, the distinguished engineer who designed it, recognized all the probable dangers which threaten such a dam, and saw no economy without absolute safety. Mr. Van Buren.

In regard to the probability of dangerous cracks, the remarks of Mr. Skinner are interesting and to the point, also those of Mr. Emery on concrete in the reservoirs of India.* It is certainly impossible to build a large mass of masonry without extensive cracks. In the case of the great Croton Dam, there will be acres upon acres of foundation, and upon it there will be built masonry as high as the pyramids. It requires unlimited confidence to claim that this structure can be built water-tight, or free from dangerous cracks and fissures. Mr. Wegmann cites two recent dams to show that masonry in large masses can be made water-tight. This may be so at first, but the effects of time are sure to develop cracks and leaks. Even absorbed water, under pressure, works its way into streams through the solution and corrosion of the obstructing material. Mr. Allen and Mr. Collingwood have suggested that only a part of the base pressure need be considered, but the difficulty is to know how much should be assumed.

As the author remembers the discussion at the meeting, and as he now reads it, Mr. Wegmann makes an important error in regard to the total toe pressure, confusing it with the ground pressure. The total toe pressure assumed is 30 000 lbs. per square foot, made up of 15 600 lbs. from the hydrostatic pressure, and 14 400 lbs. from the ground pressure. He also makes an important error in stating that the total toe pressure would be increased in case the water failed to penetrate the base of the section as designed. The omission of the base water pressure would, on the contrary, reduce the toe pressure from 30 000 to 25 000 lbs. Mr. Le Conte makes a similar error.

In adopting the theory of the middle third, the author is charged with the "perpetuation of an admitted error." Every theory is, more or less, an "admitted error," and it is necessary to cling to the imperfect theory given by the highest authorities until a better and safer one is advanced. The author knows of none. As to this theory leading to a "veritable monstrosity," it may be remarked that the theory of frictional stability leads to the same result. This is certainly not a fad. The criminal is neither of these, but the assumption of a base water pressure.

It has been suggested that the rôle in designing such a structure should be economy with sufficient safety. In the case of the great Croton Dam, sufficient safety must mean safety as absolute as skill and money can make it. New York cannot afford to take the remotest risk in its construction. The results of a failure would be appalling, the flood wave would sweep to destruction the whole western border of the city, with countless lives, buildings, docks and shipping.

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXXIV, p. 252.

Mr. Van Buren. In conclusion, the author regrets that the old-fashioned type of dam, with the flat slope up stream, has been discarded. As used in masonry, it is in harmony with true principles which forbid tension or bending strains, and utilizes an important element of safety from its shape alone, viz., the weight of the overlying water.

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Vol. XXXIV.—December, 1895.

HOLLOW TILE FLOORS, PAST AND PRESENT.

By FR. VON EMPERGER.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

The use of hollow tile for floor construction is scarcely known in Europe. The most recent European designs with flat ceilings have not yet overcome the difficulty of dividing the span into several blocks, and always use one continuous tube of tile covering the whole span, as shown by the French design, Fig. 1. This type is the same as the Liverpool "flags" used in 1853, and introduced again by Homan & Rodgers in 1885. A recently adopted German design shows divisions, but the vertical joints are bound together by imbedded iron rods, as shown in Fig. 2. It is of interest to compare these two recent designs with Fig. 3, illustrating a type proposed in 1873 by Mr. Wear L. Drake, of Chicago, Ill. Notwithstanding the 21 years which have passed, this simple design has not yet found its way across the ocean, and the reason for this is probably the cost of its production, as machine work, which has a most important part in its manufacture, is admittedly cheaper in the United States than in Europe.

The small spans used in Europe, 3 ft. at the most, involve a waste of iron and floor material, and hollow tile could not compete successfully with the concrete construction which has come into such great prominence in Europe as a fire-proofing material, especially as the cement and hand labor used in concrete is comparatively cheaper there.

All European types of hollow tile flat arches could be better termed hollow tile ceilings, as they are designed to furnish a flat fireproof ceiling, concrete being used where strength is needed. The origin of the "old style" hollow tile floor arch is probably indicated by a patent granted to Johnson & Kreischer, March 21st, 1871. The first hollow tile flooring was in the corridors of the New York Post Office and in the Coal and Iron Exchange in New York, built about 1872. Next, it was employed in the Kendall, now Equitable, Building and the Criminal Court Building, both in Chicago. It was used in 1876 in the Chicago Court House, and in 1877–78 in the Van Corlear flats in New York. It seems a well-established fact that its use was not general before the style of building known as skeleton construction was generally adopted, which cannot be assumed to be before 1889. Fig. 4 is taken from an old publication of the Phœnix Iron Company in the early eighties, and it is stated there "as usually made."

Two distinct types of arches have appeared on the market, which have been developed independently and differ in the relative position of the openings and the beams. These openings are either parallel with the beams, as in the "old style" shown in Fig. 6, or are perpendicular to them, as in the "end construction," the latter with its variety of through construction (see Fig. 5), or with separate skewbacks.

Attention is next drawn to the different arrangements of the ribs in the blocks and their development. It can be easily seen that their primary arrangement, as shown in Fig. 4, gives full justice to the engineering aim, that of an arch with a horizontal ceiling below. It is a flat arch with solid blocks, having the superfluous material removed. Mr. Drake had an alternative design (Fig. 3), with horizontal ribs, which are connected in no way with the stresses of the whole construction, and tend only to stiffen the block in its manufacture and handling. The further development has followed the same track with the exception of some well known skewback designs having curved ribs, and some designs and patents which never came into general use.

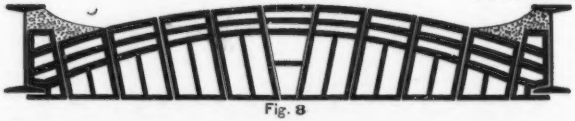

Fig. 8

Fig. 4

Fig. 3

Fig. 5

Fig. 2

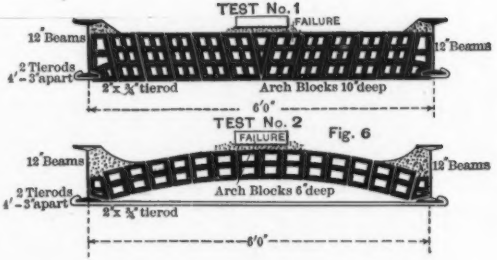

Fig. 6

Fig. 9

Fig. 1

Comparing both the Drake arrangements shown in Fig. 3, the author believes, from several independent tests, that with horizontal ribs the section devoted to an arch-like action is comparatively weak, and such a floor will, after some time, carry a load largely by its beam-like action by the help of strong mortar joints, provided they are properly made. This is also proved by the tests with arches 20 days old, made in 1891 by W. E. Cutshaw, M. Am. Soc. C. E., City Engineer of Richmond, Va. In two tests, both with 12-in. blocks, three $\frac{5}{8}$-in. ribs and a span of $4\frac{1}{2}$ ft., one arch cracked in the key at 277 lbs. and broke at 554 lbs. per square foot (arch-like action), while the other showed no crack at all until it suddenly collapsed under 1 000 lbs. per square foot (beam-like action). In two other tests of arches 40 days old, with a span of $3\frac{3}{4}$ ft., made of 12-in. blocks, as above, one broke at 770 lbs., while the other stood 1 005 lbs. per square foot, without cracking.

It is easy for anyone to convince himself concerning these two different actions by closely observing a test of a flat arch. There are two ways in which they may be noticed. In the first cracks, in the bottom of the center do not appear before there is a comparatively high load applied. This will happen when the mortar is strong enough and carefully enough placed to bear the tension stress for a while, which depends upon the setting time and the quality of the cement used and its proportion. The second method of failure will happen with a new, weak or badly made mortar. Cracks appear very soon, going higher quickly and extending up to two-thirds to five-sixths of the height, according to the depth of the part under tension. Only the second case should be considered, following the approved practice never to rely on tension in the mortar joints of an arch and to increase the section devoted to compression according to the load for which it is designed.

It can be seen that in the first case the arch-like action sets in comparatively late, and with a weak arch section the smallest crack is followed quickly by smashing the whole piece; while in the second case with a strong arch section, the results attained are always the same, and cracks may appear at the bottom which are not followed by a sudden collapse.

The arrangement of the ribs shown in the through construction (Fig. 5) is quite a departure, and needs special consideration on account of its simple manufacture. It is the same tube which European manu-

facturers advocate (see Fig. 1). The large span requires an increase of section, and convenience in manufacture necessitates cutting it in pieces with inclined joints. Only the vertical solid ribs transmit strains, the horizontal ribs serving to stiffen the block. With a proper height and width of these vertical ribs any strength desired can be attained; but as the upper part only of these vertical ribs is acting as an arch, it has necessarily to be very strong, and will require about the same amount of material as a solid flat arch.

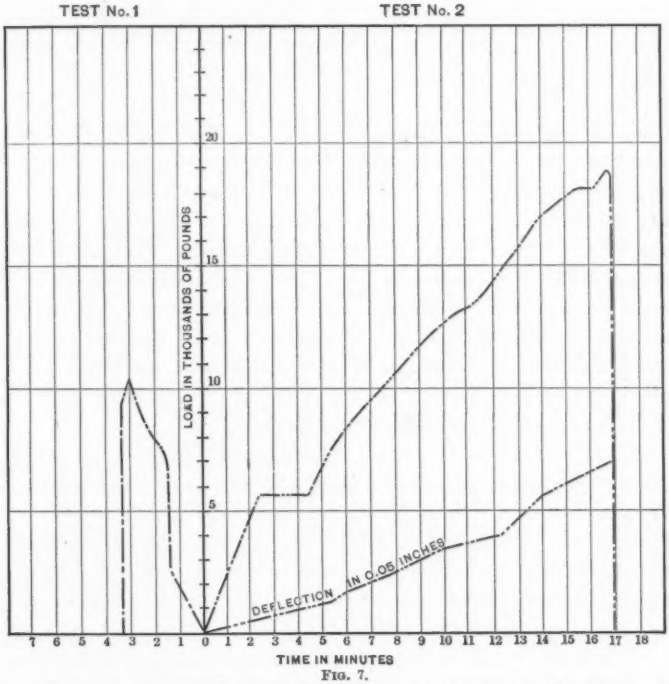
TEST No. 1
TEST No. 2
LOAD IN THOUSANDS OF POUNDS
20
15
10
5
DEFLECTION IN 0.05 INCHES
7 6 5 4 3 2 1 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
TIME IN MINUTES

FIG. 7.

The comparative design, Fig. 5, shows that such a hollow tile arch occupies only one-fourth of the section of a solid arch, and if it is to be kept within the same limits, which means to use the same amount of tile, this cannot be done otherwise than at the expense of strength. This ratio, 1 to 4, will necessarily be changed somewhat by the additional material, which stiffens this construction against eccentric loading and gives it a better side bearing than the solid arch, which should

always be tested eccentrically. Efforts have been made to strengthen this style by inserting wires, but this combination of tile and iron did not find favor for several reasons.

The author's efforts were directed to get the desired strength by a different arrangement of the ribs, and to arrange them where really wanted. In the design shown in Fig. 8 an attempt was made to satisfy this requirement by an arch-like arrangement of the ribs by increasing their tile section against the haunches, and thereby stiffening them there. This is, of course, only a development of the older style shown in Fig. 4, and departs from it in details only. The author endeavored to make some tests to substantiate his theory. Inquiries made as to the manufacture of such test arches at the most prominent tile manufactories have shown that they are all willing to have the patterns made if a large enough order is given, and that this type of block with curved ribs (Figs. 8 and 9), could be had as cheap as any other style of hollow tile floor of corresponding section. But for a small experiment only, the cost runs rather high, and the author found a cheaper and as satisfactory a way. Instead of making new blocks, he built the arch shown in test No. 2, Fig. 6, which must at least show the strength of the design as it is given in Fig. 8. The tests Nos. 1 and 2, Fig. 6, were made in connection with others of the Melan type by a hydraulic testing machine, which was built by George Hill, Assoc. M. Am. Soc. C. E., especially for that purpose.

Mr. Hill was asked to select a reliable mason, who was given the design (Fig. 6), with an order to build it in a frame consisting of two 12-in. beams connected by heavy tie-rods. The arches were built January 12th, 1895, which was a mild day, and were tested on January 25th. It may be said that the material was bought in open market from a well-known manufacturer and was put together with the greatest care.

Following is Mr. Hill's report:

"Tests were made using my hydraulic plunger testing machine of 70 000 lbs. capacity; the pressures are registered by means of a Bristol pressure recording gauge which was calibrated before and after the test, and the cards corrected for all error. Deflections were measured with an L-shape multiplying lever multiplying the deflections 10 times and correct within the small range of deflection noted.

"The application of the load was made by leveling up with sand and gravel on top of the arch a space about 1½ ft. by 2 ft., the long dimension being parallel with the skewback beams. On top of this a piece of bluestone 2 x 15 x 25 ins. was placed, and then wooden

blocking. The weight of the blocking, stones and sand was about 200 lbs., and was added to the loads recorded by the gauge.

"All calculations are made with a 10-in. slide rule and may therefore be in error about 2 per cent.

"Test No. 1.—This was a test of a side construction arch of 10-in. blocks, 6-ft. span, 4 ft. 3 ins. wide, parallel with the skewback beams, with two 2 x ¾-in. tie-rods turned over the end of the skewback beams, the section of the arch and its general dimensions being as given in Fig. 6. The arch blocks were of hard tile and represented about a fair average of the usual hard tile blocks furnished to the average building, and were much less strong than the best of hard tile blocks, in my opinion.

"The arch failed without appreciable deflection by crushing in the top horizontal piece or member of the block next to the key, or about 10 ins. away from the center, under a load of 10 600 lbs. The failure was a true crushing of the material and afterwards the first rib down went as well. The loaded area was about 1 sq. ft. The arch area was about 26 sq. ft.; load per square foot, 408 lbs., calculated on total area of arch.

"Compression in the arch can be computed in either of two ways: First, on the assumption that the effective rise is from the junction of the flange of the floor beam forming the skewback with its web up to the center of the second rib down from the top, which may be taken at 5 ins. Second, measuring from the same point to the center of the top web of the arch, in which case the depth could be taken as 8 ins. If we assume 12 ins. as the length in which the compression exists, which is probably less than the truth, we would have a total compression under the first condition of 38 160 lbs., which, distributed on 12 lin. ins., would give 1 410 lbs. per square inch, and a tension in the tie-rod of 12 720 lbs. per square inch. If, on the other hand, we assume that the load was borne entirely by 12 ins. of the top rib, we should then have a total compression of 23 850 lbs. or 2 650 lbs. per square inch. The tension in the tie-rods would be 7 950 lbs. per square inch. From what we know of this material, I am inclined to think that there must have been an initial failure at some one point which carried more than the above amount. If, instead of assuming that the arch failed first immediately under the load, we should assume that it failed entirely across the width of the top member, the fiber strains in the tile would be reduced in the ratio of 4.25 to 1. This assumption I believe to be certainly incorrect from the manner in which the failure occurred.

"The load per lineal foot of skewback, 1 260 lbs., is noticeably higher than that sustained in many former arch tests, and also notable from the fact that failure generally occurs in the skewbacks. The rate of application of the load is graphically shown in Fig. 7.

"Test No. 2.—This was a test of a segmental arch made of poor hard tile blocks 6 ins. deep of section as shown in Fig. 6. Span was 6 ins. Width along skewbacks, 4 ft. Rise, 6 ins. Skews were not well fitted and were not of a very good type. There was some filling on the haunches of the arch.

"The drawing (Fig. 7) shows the rate of application of the load, and also the deflection of the crown of the arch under it, the same spacing representing load in thousands of pounds and the deflections in 0.05 in. The failure occurred just off the center, the block failing along the line shown, one part dropping below the other and indicating a partial shearing action, under a load of 19 000 lbs. No opening at the haunches observed. Calculating the effective rise as 6 ins., the compression on the center of the arch amounted to a total of 57 000 lbs. The tension in the tie-rods amounted to 19 000 lbs. per square inch. The depression in the center amounted to 0.4 in., and the elongation of the tie-rods to about 0.12 in. The load per square foot of arch area was 792 lbs., load per lineal foot of skewback 2 375 lbs., and the effective loaded area about 1¼ sq. ft."

By comparing these figures with an equally distributed load as specified by the building laws, it will be found:

That test No. 1 carried, corresponding to the center load.....	816	lbs.	per square foot.
And in addition to that its own dead load of.....	40	"	"
Total breaking load.....	856	"	"
While Test No. 2 carried, corresponding to center load.....	1 584	"	"
And in addition to that its own dead load.....	32	"	"
Total breaking load.....	1 616	"	"

The general practice requires in these structures a factor of safety of ten, but this might be reduced in regard to its own dead load by one-half, a suggestion which is well founded by a corresponding practice in bridge building. Then with a dead load of 75 lbs. per square foot for finished floor (375 lbs.) there remains for Test No. 1 a balance of the breaking load of 481 lbs. per square foot, or a safe live load of 48 lbs. per square foot, and for Test No. 2, 1 241 lbs. per square foot, or a safe live load of 124 lbs. per square foot.

The smallest load the building laws allow is 70 lbs. per square foot in dwellings, so that the floor section in Test No. 1 did not come up even to this moderate requirement, while Test No. 2, having the same tile section, is adapted to bear a load of 124 lbs. per square foot, and is adequate to carry any weight expected in ordinary buildings, especially as the strength would naturally be still increased by an arrangement such as is shown in Fig. 8.

The breaking load of Test No. 1 of 856 lbs. per square foot is about the same as other experiments have shown by careful and repeated testing, although reliable tests are scarce, and most of them are not arranged so as to prove of scientific value.

To illustrate by a single case how even a very faithful experimenter may succeed in getting deceptive results, reference is made to the tests of flooring carried out in Denver in 1890.*

The tests were made by loading the arches with a mass of loose bricks. A uniformly distributed load by such big piles should not be used in these tests, because it represents a solid block which cannot follow the deflection of the floor below it, so that by an unnoticeable deflection the center relieves itself and the load spreads to the haunches. Even if this is not the case, it is never known how the load is really distributed. For this reason, as well as on account of the great convenience in handling, it is believed that Mr. Hill's method will create a turning point to sound practice in testing and consequently in floor construction itself.

The present practice is that an engineer's duty ends after designing the iron-work, and then the architect writes the words "hollow tile floor" across the design. How this hollow tile floor shall be made, how deep the block, and how thick the tile section, does not seem to be considered in connection with the span and the floor load as is done in regard to iron, but is simply a matter decided generally by the cost, sometimes by chance, but always for the convenience of the manufacturer who gets the contract.

The fact that this style of flooring is now used for warehouses and even for floors intended for the heaviest moving loads in manufacturing plants, and that this is done with practically the same 8-in. blocks employed in office buildings indicates that the building laws should prescribe an adequate section according to the load and span, and the

*See the *American Architect*, Vol. XXXI, p. 195.

author submits the following draft of such requirements as a basis for discussion:

	Fig. 9.		Fig. 8.	
Live load. Pounds per square inch.....	70	100	120	150
Largest permissible span. Inches.....	5	6	6	6
Size of block. Inches	8	9	10	12
Number of ribs.....	2	2	3	3
Thickness of ribs at crown. Inches.....	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{3}{4}$
Thickness of ribs at haunches.....	Increasing 33 per cent.			
Dead load of tile arches. Pounds per square foot.....	30	35	40	45

There is already a reduction of dead load in case of small live loads, and it is believed that still smaller dimensions might be warranted by further tests.

It is a remarkable claim for the present style of flooring that by the use of horizontal ribs the arches are not confined to certain spans, but that one and the same block can be so arranged as to fit in any span desired without increasing the costs of setting and manufacture. It is important to state, that by a special arrangement, which would take too much space to explain, the author has succeeded in making the same claim in regard to the curved ribs. There is, of course, no difficulty in giving those blocks a horizontal top, as shown in Fig. 9. In either case there is no need of concrete, except in the haunches in Fig. 8, while it is a well known secret that the concrete cover of the present flat arch is sometimes the only thing which gives it the necessary strength, and that the tile section sometimes used in public buildings is of surprising flimsiness.

There is now a deplorable difference between the tile which the manufacturer uses for his own tests and those bought in open market. There is a further difference in the quality and quantity of the cement used in tests and in regular construction. The mortar used in tests should be of the poorest quality that will be employed in actual work, and it would be the best endorsement of an arch, if, with the use of sand alone, the tests give satisfactory results.

It seems to the author that there is an improvement in the type shown in Figs. 8 and 9, in that the least dimensions are distinctly specified according to the load, and, further, that it is known which

part carries the weight and is essential to strength. It is then easy to see that this part, the arch proper, is composed of faultless material. The remaining part, the suspended ceiling, may have cracks and breaks, which do not matter, and perhaps it could be made either of cheaper material or, at least, of the thinnest tile, to reduce the weight and cost. As no reliance is placed on the joint, the use of a high-grade cement, mixed to give a strong mortar, is unnecessary.

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Vol. XXXIV.—December, 1895.

MOVING TWO 36-IN. WATER MAINS WITHOUT SHUTTING OFF THE WATER.

By E. C. MOORE, Jun. Am. Soc. C. E.

READ OCTOBER 2D, 1895.

WITH DISCUSSION.

The Metropolitan Street Railway Company of New York secured the franchise for a cable road through Lexington Avenue from Twenty-third Street north to the Harlem River, and the author has been the division engineer in charge of its construction. The fact was recognized early that one of the most serious difficulties to contend against was presented by the substructures already in the street. With this fact in view an attempt was made to get an accurate record of exactly what was in the street, by looking up the maps and records of the various gas companies, the city water department and the subway company.

The maps of the gas companies were thoroughly unreliable as to location, as was also the water map; those of the subway company were a little better, but not infallible, so at an early stage in the construction these maps were entirely discarded and no calculations

were made for the moving of obstructions until they were actually encountered in the work. It was found more convenient and less expensive to move obstructions after the excavation had been completed, and the extra expense of a second digging was saved. More obstructions were encountered at intersecting streets than elsewhere, and it was frequently necessary to move two or three outlying pipes or subways in order to get the room required for moving the one which was in the way.

While this system worked very well on Lexington Avenue, where there were no cars to be taken care of on side tracks, it would not answer when the work was extended through Twenty-third Street, where there were not only a large number of cars to be taken care of on the side tracks, but a heavy traffic through the street as well. Moreover, the business houses along the line had to have entrances to handle their wares. Provision was also made for two cables for Twenty-third Street proper, in addition to the Lexington Avenue cables, and the construction problem was therefore a complicated one. All the mains which were parallel to and within the tracks already in use were replaced by new mains, laid on the outside, and in some cases new mains were laid on one side of the street, when there was already a main on the other side, in order to get rid of the house connections, which usually ran too high for the cable construction.

The success of these operations was encouraging and furnished ground for the belief that it would be possible to move two 36-in. water mains in Broadway, between Twenty-second and Twenty-third Streets without shutting off the water, which would not be permitted by the Department of Public Works. These mains were laid in 1839, and had joints every 9 ft.

It was decided that the pipes, which had to be dropped a maximum distance of 2 ft. and moved laterally 1 ft., should be lowered by chains and turn-buckles swung from yokes placed over them at each joint. The yokes were made of 8 x 12-in. yellow pine, the posts being 8 ft. 6 ins. long set on a wooden foot block of 5 x 12-in. yellow pine 2 ft. long, and the cap piece was about 11 ft. long, being 9 ft. between the posts.

The posts were sawed off square, and, after they were set up in place, the cap piece was laid on top of them without framing, then cap and post were fastened together by a cleat of 2 x 9-in. spruce about 2 ft.

long, which was nailed to their sides, one being used at each post and on opposite sides of the bent. Then two diagonal braces about 3 ft. long of 2 x 9-in. spruce were nailed diagonally from the cap piece to the opposite side of the post.

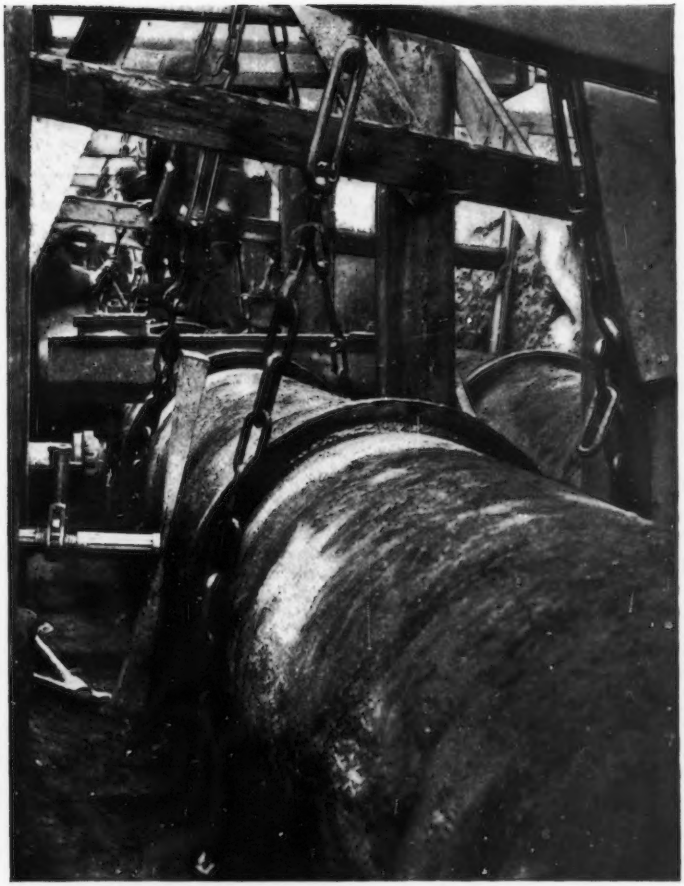
As soon as the first bent was set up and braced from the ground, the next bent was set up, and the two were fastened together by nailing two pieces of 2 x 9-in. spruce on top, reaching from one bent to the other at the ends, and so on until twenty bents were in place.

Then the chains, which were of ¾-in. round iron with links 7 ins. long on the inside, were put in place (see Plate VII). The chain which went over the yoke was 5 ft. long, and that which went around the pipe was 11 ft. long. There were two turn-buckles at each bent, so that the chain would not have to slip around the pipe or the timber in lowering. Each turn-buckle was of 1¼-in. iron, with hooks on each end, and could lower 12 ins. without changing hooks. The hooks were made to just fit in the links of the chain, and when the pipe was dropped as far as the turn-buckle would go, each turn-buckle was taken off in turn and the hooks placed in links nearer the end of the chain, when the lowering proceeded as before. Everything was made of sufficient strength to allow of every other turn-buckle being taken entirely off without blocking the pipe. This proved of great advantage, as much time would have been lost if it had been necessary to lower the pipe down on blocks in order to change the turn-buckle.

When the lowering of the pipe began, it was decided to move it sidewise at the same time. This was accomplished by hanging the chains over the cap piece a few inches to one side of a perpendicular through the center of the pipe, and putting a brace of 4 x 6-in. spruce from the end of the cap pieces to the bank, to prevent the carrying over of the top of the bent toward the pipe, instead of the desired movement of the pipe in the opposite direction. At the same time, a brace of 2 x 9-in. spruce was added at each end of each bent, running diagonally down from the top of one bent to the bottom of the next, and in opposite directions at the opposite ends of the bents. This was to take care of the torsional strain caused by the cap piece being at right angles to the pipe. By moving the chains along the cap piece a few inches at a time, it was found that when the pipe was down it was exactly in position sidewise also.

By beginning at one end, it was possible to lower the first 100 ft.

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIV, No. 768.
MOORE ON MOVING TWO 36-IN. PIPES.



of the pipe into position, and to take down and move ahead the first eleven bents; then, as the pipe came into position, each bent was taken down and moved ahead again. The two pipes lay parallel to each other and 5 ft. apart from center to center. One pipe was lowered at a time. There was one hydrant connection on each pipe; this was cut off outside its gate, and a clamp of wood was fitted around the pipe and a long arm attached which reached out to the cut in the connection, and gave it support at that point. Clamps were put around the joints in the hydrant connections, to prevent any possibility of their blowing out.

It was found that almost no calking was necessary during the lowering of the pipes; there were occasional small leaks, but they generally stopped themselves as the pipe was lowered and the joint had a chance to come back into place. It was thought desirable to do no more calking on the pipe than was absolutely necessary until the work of lowering was entirely finished; then all the joints were gone over and put in good condition.

As it was desired to push the work as fast as possible, a connection was made with an electric lighting cable, and eight arc lights and about twenty incandescent lights were set up, and the work proceeded night and day. Excavation began on June 19th, and the work of putting in the bents over the mains began on June 22d. Both pipes had been moved and were in place June 29th, and within another week the trench was filled and paved over, except where the line of the excavation for the cable tracks crossed.

The pipes were moved the extreme distance for 100 ft., and 100 ft. was allowed at each end of this distance for the gradual swing, so as not to strain the joints too much by short bends. This made a total length moved of about 300 feet for each pipe.

The approximate cost of the moving was about as follows:

40 1½-in. turn-buckles with hooks on each end..	$102 40
320 lin. ft. ⅞-in. B. B. chain, 7-in. links.....	102 95
Lumber, including bridging and wedges.....	298 74
Electric lights.....	175 00
City inspectors.....	463 62
Engineers.....	109 00
Labor, carts, etc.....	4 250 00
Total.....	$5 501 71

The foregoing cost may be considered somewhat high, but while the main object sought was the completion of the work in the shortest possible time, every effort was made to avoid unnecessary expense. Night work adds very materially to the cost of such undertakings, because a man cannot do as much work at night as in the day time, and there is also more opportunity to shirk. The cost of providing lights and other conveniences and safeguards must be considered.

DISCUSSION.

Mr. Wegmann. EDWARD WEGMANN, Jr., M. Am. Soc. C. E.—The speaker stated that there were other cases than those mentioned in the paper in which 36-in. mains in New York City had to be shifted and lowered to a considerable extent.

When the old Croton Aqueduct was constructed, the masonry conduit terminated at a receiving reservoir located in what is now Central Park. From here two lines of 36-in. mains, laid in Eightieth Street to Fifth Avenue, and then down this avenue, conveyed the water to a distributing reservoir at Forty-second Street. After the mains had been laid, changes were made in the grades of Fifth Avenue, which necessitated the lowering of the pipes at some points. The first case of this kind occurred in 1850, at Murray Hill, near the distributing reservoir. At this place the pipes had to be lowered 16 ft. The Commissioners of the Croton Aqueduct Department decided at first to remove the mains until the avenue had been excavated to the new grade, and then to relay them. If this had been done, the only connection between the receiving and distributing reservoirs, for the time being, would have been a line of 30-in. mains that had been laid the preceding year in Third Avenue. Mr. E. W. Tracy, one of the engineers of the Aqueduct Department, who subsequently became chief engineer of the Croton Aqueduct, proposed a practical plan for lowering the pipes in place, which was accepted and successfully carried out under his direction.

At the place in question the material to be excavated was hard pan. Small drifts, located about 8 ft. apart, were excavated under the mains. In these drifts blocking was placed to support the weight of the pipes. After this had been accomplished, the material remaining between the drifts was excavated. By using jack screws, the blocking was then gradually removed and the pipes lowered to the new grades, the additional length being obtained by the pipes drawing a trifle out of the joints. Only one line of mains, from which the water was shut off, was operated on at a time, the other line being kept in service.

Mr. Wegmann.

Another case of this kind occurred in 1856, on Fifth Avenue, between Forty-seventh and Fifty-fourth Streets. In this instance, the two lines of 36-in. mains were lowered a maximum distance of 11 ft. without the water being shut off from either line. As the excavation had to be made in solid rock, the pipes had first to be shifted to one side. This was accomplished after the avenue on the east side of the mains had been excavated to the new grades, by placing under the pipes wooden saddles that rested on greased skids. The mains for a length of 1 300 ft. were then moved over to the east, by means of jack screws, a maximum distance of 10 ft. After a pipe trench had been excavated in the middle of the avenue, the pipes were moved back to blocking which had been placed in this trench. The lowering of the mains was then accomplished in the manner explained in the first case. During the shifting and lowering of the pipes but very little calking of the joints was required. The work described, which attracted considerable attention at the time, was performed under the direction of the late James P. Kirkwood, Past-President Am. Soc. C. E., subsequently chief engineer of the Brooklyn Water-Works.

In 1870, the two lines of Fifth Avenue mains had to be lowered between Sixty-eighth and Seventy-second Streets a maximum distance of 4 ft. The work was performed successfully under the direction of Gen. George S. Greene, Past-President Am. Soc. C. E.

Mr Owen.

JAMES OWEN, M. Am. Soc. C. E.—While the speaker had no experience in moving large water mains, he had lowered many small pipes without shutting off the water. The first he undertook to move was an 8-in. main that was to be dropped a number of feet. Elaborate preparations were made for doing the work slowly and uniformly, but at the last moment something gave way and part of the line of pipe fell to the bottom of the trench. It was with considerable anxiety that he descended to examine the condition of the main, but when it was found to be intact and tight, he experienced a change of feeling and has ever since felt confident of success in such undertakings.

Mr. Platt.

JOSEPH C. PLATT, M. Am. Soc. C. E.—In the summer of 1894 one of the 36-in. pipes of the Brooklyn Water-Works was uncovered, and a 12-in. hole cut on one side in order to connect a pipe of that size with it. This 36-in. pipe was one of the original mains of the system. It was made in Scotland and laid as early as 1859, and was probably well covered with the genuine Dr. Angus Smith's coating.

The piece which was removed was free from tubercules, pitted holes and all other evidence of deterioration.* In fact, it appeared as good as many lengths which have lain in a pipe yard for a year. The iron was strong and clean, a good pipe or machinery iron, and quite different from much that is of later date.

* This piece is now (October, 1895) on exhibition at the office of the Water Purveyor of Brooklyn.

Mr. Platt. It has been stated by those in charge of the Ridgewood station of the same water-works, where the water is pumped before it runs through these 36-in. pipes, that the pump cylinders and chambers handling this water are so deteriorated that the iron will not hold a tap bolt, but that it is necessary to use bolts running through the metal and a packed washer on the inside, whenever it is desired to make an attachment to the parts in question. The speaker did not attempt to decide whether the great difference in the lasting qualities of the two irons was due to the fact that the pipes were coated and the machinery was not, or to a difference in the metal originally, or to the frequent vibrations of the pumps, from which the pipes are free, or some other cause.

CORRESPONDENCE.

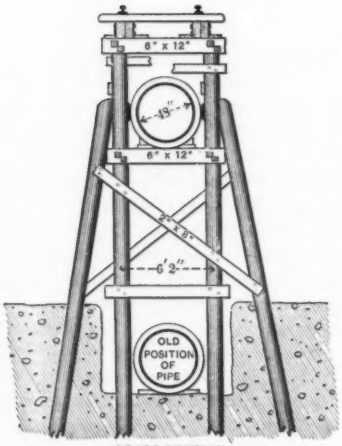
Mr. Brackett. DEXTER BRACKETT, M. Am. Soc. C. E.—The writer has had occasion to move a number of large water mains, and as the methods adopted have been somewhat different from those described by the author of the paper, a description of the work may have some value to others in similar undertakings.

In 1885–86 the grade crossings of Brookline Avenue and Beacon Street with the Boston and Albany Railroad were abolished, and the streets carried over the railroad. Each of these streets contained one of the large mains supplying the city of Boston, which, if not raised, would have been buried 20 ft. below the surface near the railroad, the grade running down and meeting the old grade of the street about 900 ft. from the bridge in either direction. Both pipes were raised in substantially the same manner, so that a description of the method of raising the 48-in. pipe in Beacon Street will suffice for both.

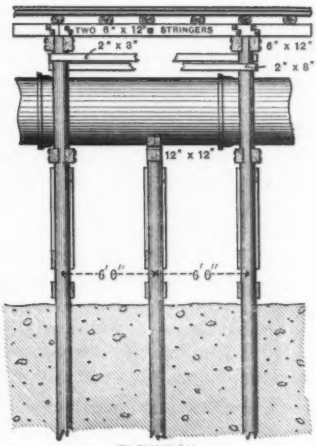
The bridge was first built, the pipe being left in position and the masonry of the bridge abutments built around it. Piles were then driven at intervals of 6 ft. on each side of the pipe for the entire distance to be raised. Alternate bents of the piles were capped 6 ins. below the new grade of the street, with 6 x 12-in. double girder caps. The intermediate bents of piles were cut off about 12 ins. below the new grade at the bottom of the pipe, and fitted with 12 x 12-in. caps, which, however, were not placed in position until after the pipe had been raised.

The earth was then excavated from around the pipes, which were raised by means of screws 8 ft. 9 ins. long and 2.5 ins. in diameter, placed 12 ft. apart. These screws passed between the 6 x 12-in. girder caps, and were supported by a cast-iron plate 15 ins. square, which rested on the caps. The distance between the chains, which were placed around the pipes, to the bottom of the screws, was made up

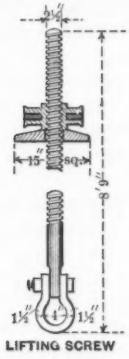
Mr. Brackett.



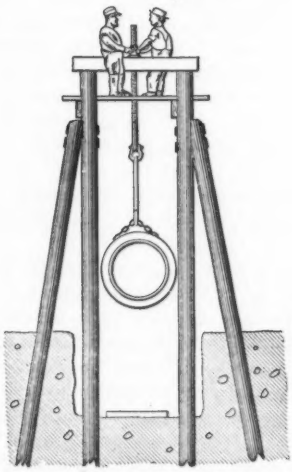
CROSS SECTION



ELEVATION



LIFTING SCREW



CROSS SECTION SHOWING METHOD OF RAISING PIPE

FIG. 1.

Mr. Brackett. where necessary by long hooks made of 1-in. steel. The dimensions of the pile bents and lifting screws are shown in Fig. 1.

When everything was in readiness, the pipe was cut at the bridge abutment, and the entire length of 900 ft. to the foot of the grade was raised in about four hours. The 12 x 12-in. caps were then placed in position and the pipe joints redriven. The piles which had been used to support the lifting screws were girder-capped under the pipe, but the wedges between the caps and the pipe were not driven until the street had been filled to the grade at the bottom of the pipe, for the reason that the filling was done from cars run on the trestle, and the jar caused by running and dumping gravel trains would have been likely to cause leaks in the pipe. But one-half of the 1 800 ft. could be raised at a time on account of the number of lifting screws available. These screws have been in use by the water department for many years, and have proved of great value on many occasions.

In 1889 about 1 000 ft. of 48-in. pipe was lowered, and in this case a much simpler and cheaper plan could be adopted. The pipe was lowered a maximum distance of about 2 ft. in the following manner: The trench was first excavated to the depth of the bottom of the pipe. Cross trenches were then dug every 12 ft., in each of which was placed a blocking, the top of which was at the new grade of the bottom of the pipe. Laborers were then placed on each side of the pipe, and the earth gradually removed from beneath the pipe, thus lowering it to the required grade. The time occupied in lowering 1 000 ft. of pipe and redriving the joints was but about eight hours. Although the gates on the line were closed, the water was not drawn from the pipe, and the writer would have had no hesitation in maintaining the flow through the main if it were necessary. In this case no screws or blocking were used.

On another occasion a 40-in. main crossing a bridge was lowered while in use, but in this case the pipe was supported and lowered by screws.

Mr. Moore. E. C. MOORE, Jun. Am. Soc. C. E.—While the moving of large water mains, both with and without the water on, and sometimes the pressure, is no novelty, yet each particular case presents its own problem, and it is quite possible that the most careful canvass of past methods may not furnish an exact parallel. A certain amount of originality therefore becomes a necessity.

This was especially the case with the work described by the author. In the first place, the number of lineal feet of pipe to be moved was comparatively very small. The cost of excavation and preparation for moving was very large, due to the fact that the work was located in one of the busiest points along Broadway. The matter of public convenience had to be given serious consideration, and, therefore, it was necessary that the swing of over 2 ft. be made in the shortest distance

in which it could be safely done. After careful calculations, it was decided that eleven joints on either side of the part moved would furnish the necessary deflection without dangerously pulling any of them. The conclusion proved to be a correct one, for some of the joints were certainly pulled all that they would safely stand, and yet there was no accident to any of them. Mr. Moore.

As to the method employed, there were several reasons for its adoption In the first place, some of the possible consequences of a break in the pipes had to be considered, the most serious of which would be a fire in the downtown district, in fact almost anywhere below Fourteenth Street, in the proximity of Broadway, in which district one of the mains moved furnishes almost, if not quite, the sole supply. It would have been hardly possible to repair any break and get the water turned on in less than four hours, even if everything went smoothly and every one of the old gates worked perfectly, which is hardly probable. Any one of the dozen which would have to be shut and opened again failing to work might make the four hours twenty-four. There is no use in wasting figures on the possible amount of damage a conflagration might do in the meantime. Going from the possible to the certain, a break of any appreciable extent would have undermined the Broadway cable tracks, flooded all the adjacent cellars and blocked all traffic in Twenty-third Street and Broadway. Therefore it was considered necessary by the author to take every precaution possible and spare no expense which might add an extra safeguard, and it is with all these facts in view that the author considers the successful carrying out of this work worthy of record.

In the discussion there seems to be an idea prevalent that all past attempts of this kind have been successful and therefore the actual danger was very small, but such is not the case. While the author has not looked up the records of work of this kind he knows from his own experience that it is always dangerous to attempt to do anything with old pipes, for age seems to make them very hard and very brittle. In Lexington Avenue the 6-in. water pipe, which had to be moved all along the line, frequently broke and sometimes would not admit of any movement whatever. Mr. G. W. Birdsall had the same experience in moving old mains in the city, it frequently being the case that a very small movement would split the hubs. Therefore the author considers it certainly unsafe to attempt the moving of a main with the pressure on, without first being sure that the pipe is in a condition to stand the strain.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXXIV.—December, 1895.

TESTS OF FIRE-PROOF FLOORING MATERIAL.*

By GEORGE HILL, Assoc. M. Am. Soc. C. E.

READ DECEMBER 4TH, 1895.

The discussion of Mr. Fr. von Emperger's paper on "The Development and Recent Improvement of Concrete-Iron Highway Bridges" † brought out very clearly that great numbers of tests of the materials entering into building construction have been made, and the characteristics of the materials very accurately determined; yet the subject of floors had received relatively but little attention, and that not of a kind to indicate with any certainty how strong any particular form might be, what the principles were which governed the construction of a floor, or how the materials in use in this country could be so shaped as to develop the maximum of strength with a minimum of

* The Discussion and Correspondence on this paper will be printed in a subsequent number.

† See the *Transactions* of the American Society of Civil Engineers, Vol. XXXI, p. 457.

material. These facts were already known to the author from investigations which he had made in order to determine with some degree of accuracy what the factor of safety of floors was, and what the most advantageous disposition of the material might be, taking the shapes which were on the market. He therefore determined that, after reviewing the subject generally, bringing together as many published tests as possible, he would make a new series of tests from the engineer's point of view, to determine something positive regarding the subject. He had in mind at the time principally those forms of floor construction which furnished a flat platform to work on above, a flat ceiling below, and had for their principal constituent clay in some form or other, and therefore consulted with about a dozen manufacturers of fire-proof floor arches. These at first demurred and were very indifferent, then became interested, and finally all promised hearty cooperation, and that they would build nine or more arches according to certain rules which the author would lay down and under the supervision of his assistants, which arches should be tested to destruction, it being understood that their only expense would be for building the arches and furnishing men to handle the testing machine when breaking them. Mr. von Emperger also desired to have the Melan system represented in such a test, and, consequently, instructions were prepared in conjunction with him for the building of a series of Melan arches.

After settling these preliminaries satisfactorily, the author designed a hydraulic transverse testing machine, which was built for him by the Watson & Stillman Hydraulic Works. Considerable time elapsed while these matters were under discussion and the machine being built. When the machine was approaching completion, several of the manufacturers who had promised to build arches found that their interest had diminished, or their cash was threatened by the prevailing "hard times," or something else had happened, to such an extent that they neglected to build the arches altogether. The Melan system arches were, however, built during October, 1894. The Raritan Porous and Hollow Brick Company built several series for test in 1895, and quite a number of arches were furnished by the Staten Island Porous Terra Cotta Lumber Company for test in 1894 and 1895, and it is hoped that certain other manufacturers will build arches and have them tested prior to the discussion of the paper, at which time data

will be given concerning any additional tests which may have been made.

Testing Machine.—A very thorough study of the tests which have been made heretofore of fire-proof floors shows that one of three forms of loading has been generally adopted: 1, pig iron; 2, piles of brick; 3, hydraulic pressure as used by J. B. Johnson and Edward Flad, Members Am. Soc. C. E., in some tests which they made in St. Louis.

In the comments in every case following these tests where brick or pig iron was used, mention was made of the difficulty encountered in handling the loads where the arches possessed high resistance. In the arrangement used by Messrs. Johnson and Flad, it is self-evident that unless water at high pressure can be obtained, the apparatus is of limited utility, and it is further evident that it would be applicable to a very small range of spans. There was also noted in all cases a difficulty in observing satisfactorily the way in which they acted immediately before failure, and the author therefore determined to design an apparatus that would be of general use for testing floors in buildings over any of the usual spans, would be portable, would itself write the record of the application of the load, and would apply the load in such a way as to make it feasible to examine minutely the arch under test.

The machine was designed and built in the following manner (see Plate VIII): A 15-in. I beam, 80 lbs. per foot, 7 ft. 9 ins. long, was used as the principal member. It was supported above the I beams, which form the skewbacks of the arches by means of two pairs of wrought-iron clamps, shown at *a* in Plate VIII. On one end of the large beam a small hydraulic pump *b* was placed, and connected to the pump by a long coil of copper pipe was a ram *c* for putting pressure on the arch. The pump was $\frac{7}{8}$ in. in diameter of piston and $2\frac{3}{4}$ ins. stroke, and one man could get a pressure of 1 500 lbs. per square inch, or 75 000 lbs. on the arch, quite easily. It was one of the standard Watson & Stillman pumps with brass valves, and never gave the slightest trouble. Once in the middle of the test some sticks became mingled with the oil which was used instead of water in order to prevent freezing, and one of these small sticks lodged under the valve; the valve was taken out, cleaned and replaced without any difficulty whatever. The ram was of 7-in. stroke, about 8 ins. in diameter and exactly 50 sq. ins. in

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
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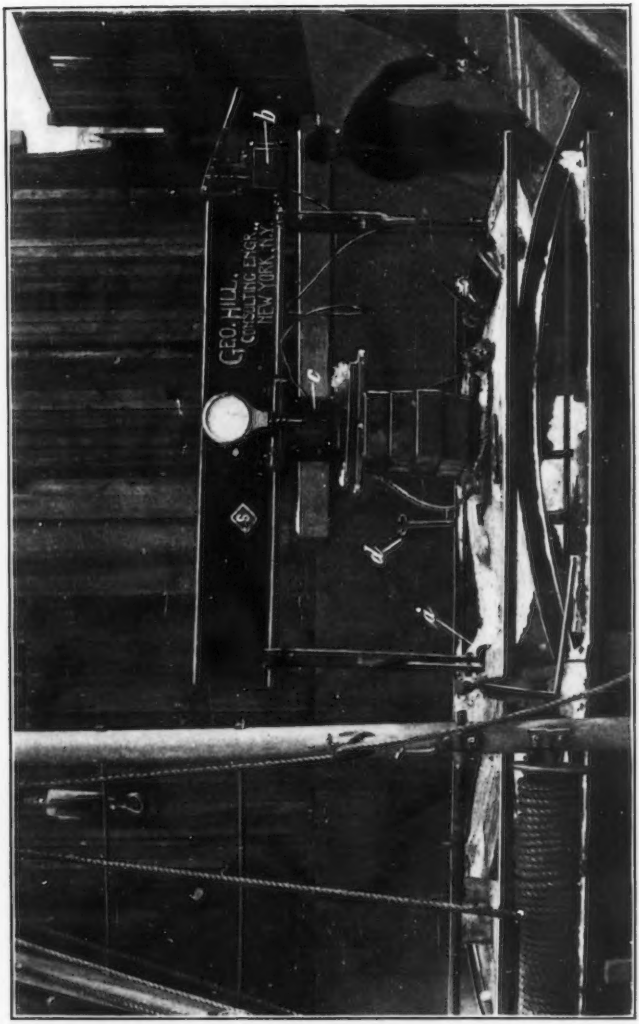
HILL ON FIRE-PROOF FLOORING MATERIAL.



area, and had the bottom crowned about $\frac{1}{8}$ in. It was set in a cast-iron body and was packed with leather. It was arranged to hang on the under flange of the 15-in. I beam by means of a couple of clamps, which could be tightened on it, and could be slid along to any desired position on the beam. To the upper part of the chamber of the plunger a set of three valves was connected, to which three Bristol pressure-recording gauges were attached. One had a range of 3 000 lbs., one of 1 500 lbs., and one of 400 lbs. per square inch. The readings were, therefore, to be multiplied by the area of the piston, or 50, to get the actual pressure exerted. The dials were rotated by clocks, making one revolution per hour, and so both the amount and the rate of the application of the load were recorded graphically. Two gauges were used in conjunction at all times, thus checking one another. In the tests made they always agreed closely.

In plotting the tests, the curved co-ordinates were made rectilinear and enlarged about 30%, so as to facilitate the reading of the results. The clamps were first made of good wrought iron, bent to shape, but under a pressure of 50 000 lbs. these clamps broke at a point marked *d* in Plate VIII, the metal going clean off, but with a good and rather fibrous fracture. They were then increased to 3 x 1$\frac{1}{4}$ ins., and under a pressure of about 70 000 lbs., eccentrically applied, as will be seen in the loading of Test No. 87, they snapped again. They were finally made $\frac{3}{4}$ x 6 ins. at the clamp and $\frac{3}{4}$ x 3 ins. higher up, and these have sustained a load of 125 000 lbs. without signs of strain. With the exception of these clamps, the machine worked so adequately that no changes of any kind suggest themselves as desirable. For handling this machine either a stone-setter's derrick or a small boom derrick or a chain block carried by a stand similar to the four-legged stand used by the weighers in the street and mounted on small wheels may be used, and a small screw jack is needed to push back the plunger after it has been extended its entire length. With such an arrangement a recorder and two men can readily break a series of twelve arches in a day, making all necessary observations.

Melan System Arches.—Mr. Fr. von Emperger, representing the Melan system in this country, had a series of arches built in accordance with the system in the author's presence, between October 10th and 27th, 1894, for the special purpose of having them tested to destruction and their strength ascertained. These arches were built of

Mannheim Portland cement, a rather poor building sand, composed of various sizes of stone, from fine sand up to round gravel stones of ¾ in. diameter, and rather dusty broken trap rock, of a size that would go through a ¾-in. ring and smaller. The arches were composed of about two-thirds of coarse material, made by mixing one pail of Portland cement, two pails of sand and four pails of stone together, and about one-third of a finer material, which was made by mixing one pail of cement with two pails of sand. The cement was of such a fineness that 98.8% passed a No. 50 sieve, 92.5% a No. 74 sieve, 88.9% a No. 100 sieve, and 67.1% a No. 200 sieve. Two briquettes of three parts by weight of sand to one part of cement had a strength of 250 and 256 lbs. per square inch when 28 days old. The amount of sulphate of lime present was 2.55 per cent.

The arches were made by putting in between 12-in. I beam skewbacks three curved ribs, one in the center and one at each end, of either I or T section. These were struck to a true circle of about 8 ft. radius. The shapes were wedged to the skewback beams by means of thin steel wedges and came down as low as the fillet joining the web and flange. Below the beams was placed a board center, relatively tight, made of stripping, the center coming close against the bottom flange of the beams. Starting at one skewback a certain amount of coarse stuff was put in for about two-thirds of the thickness of the arch, and then fine stuff was filled in to the level of the top flange of the small curved beams. Then a rammer was applied tangentially to the soffit of the arch, compressing the material along the lines of the arch; the amount of compression will be seen from the tables of the tests, where the number of cubic feet of material as measured by the buckets, each ingredient being separately measured, is given, and also the cube of the arch after being rammed. First, one side was built up almost to the center, then the other side; finally the key was filled in, and rammed vertically. After the arch all the way over had been built, the haunches were filled in on the lines shown in Figs. 3 and 4 with coarse material, and then a little fine material was spread over the top in order to give a smooth and even surface. Considering the manner in which the work was done and the character of labor employed, the author thinks that it would be fair to assume that this was done with about the same care that would be employed in a building where reasonable supervision was exercised on the part of the contractor. The men who were em-

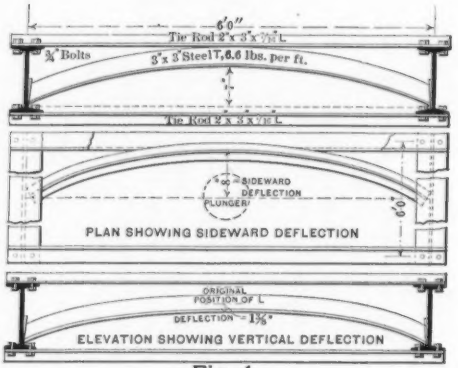


Fig. 1

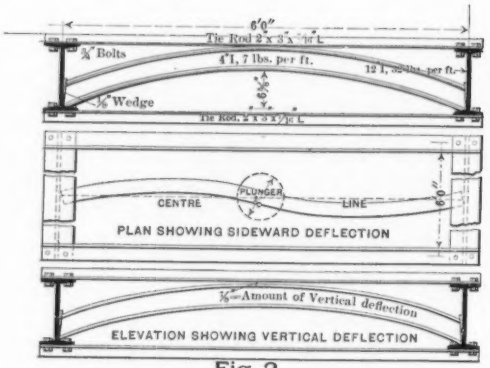


Fig. 2

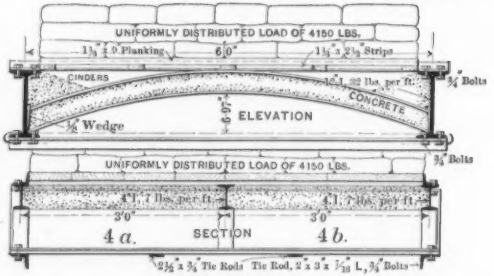


Fig. 3

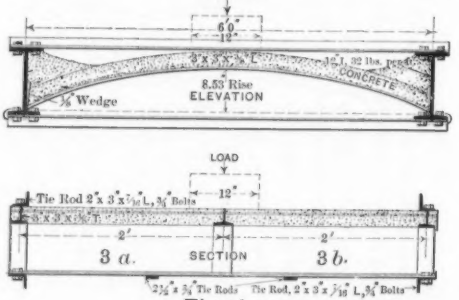


Fig. 4

ployed to do the work were not in any sense experts, and therefore could not be expected to get the very best results from the material. All of the arches were completed by October 27th, and were then permitted to set during a rather severe winter until January 25th, 1895, when the general testing began. During the time immediately after the arches were built, there was no frost to injure them, the minimum temperatures as reported by the Weather Bureau in New York being 32° Fahr. on November 7th, 9th, 11th and 15th, and 28° Fahr. on November 12th.

Test No. 76.—This was of a steel T section of ordinary commercial make, and presumably of 16 000 lbs. safe working strain, 32 000 lbs. elastic limit, and was intended to ascertain the strength of the section alone. Upon the application of the load, the crown flattened out to the position shown in Fig. 1, and then sagged off to one side, continued pumping resulting only in increasing the horizontal deflection and decreasing the pressure. Fig. 17 gives the curve of loading during this test, the maximum load being 14 000 lbs. Deflection began at 9 500 lbs.

Test No. 77.—This was of a steel I beam of the size shown, and for the same purpose as Test No. 76. Under test, the beam first flattened at the crown; then twisted to right and left so that in plan it formed an elongated letter S, as shown in Fig. 2, and then the top and bottom flanges wrinkled or buckled under the pressure, the bottom flange buckling the more. This indicated that the entire section was under compression, except perhaps a small section of the top flange near the crown. Fig. 18 gives the curve of loading during this test. The maximum load was 14 000 lbs.

Test No. 78.—This was of an arch eight days old, built as shown in Fig. 3, which was leveled up with cinders in the usual manner, but without any concrete or grout poured in. It was then stripped with 1¼ x 2½-in. strips placed parallel with the skewback beam and set about 16 ins. on centers, and then planked with 1¼-in. plank so put on that the plank bore directly on the cinders. The arch was uniformly loaded with flagstones; each stone was measured and its weight determined by allowing 160 lbs. per cubic foot or 0.092 lb. per cubic inch. The purpose of this test was to determine how quickly the material could be used with safety as a working floor; no deflection, cracking or change of any kind in the material was observed. Table

No. 1 gives the weights per square foot, etc., while the composition of the arch in detail was as follows:

DETAILED COMPOSITION OF ARCH OF TEST No. 78.

Arch.	Cubic feet. Coarse.	Cubic feet. Fine.	Compacts.	Cubic feet. Coarse.	Cubic feet. Fine.	Compacts.
4 *a*.....	0.908	0.908	0.833	0.908	0.681	1.675
	1.362	0.908	0.833	1.135	1.362	Key.
	0.908	0.681	0.833	1.815	.....	Leveling.
	1.362	0.908	0.791	9.760	6.243	Total.
	1.362	0.795	0.791	.....	.....	.....
4 *b*.....	0.454	0.908	1.000	0.908	0.908	0.75
	0.908	0.908	0.833	0.908	0.908	Key.
	0.908	1.135	0.666	1.815	0.908	Leveling.
	1.815	0 908	0.833	1.815	.....	.....
	1.135	0.908	0.917	10.666	7.491	Total.

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 4 *a* AND 4 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	6.8	6.8	3.4	3.4	Cement.
Stone.....	13.6	...	1.36	1.135	Water.

Test No. 79.—This was made of the same arch immediately after Test No. 78. The load of flagstones was practically concentrated over a length of 26 ins. of one of the 2½-in. strips; no deflection, cracking or change of any kind was observed. The center of application of the load was at the crown of the arch midway between the center and one of the side 4-in. beams.

Test No. 80.—This was of the same arch when four days older. Additional loading was put on so that the transverse strip at the crown carried 7 625 lbs. for 40 ins. of its length, and 5 320 lbs. for 26 ins. There was no appreciable deflection, sign of crack, or other disturbance noticed in the arch.

Test No. 81.—This was of a Melan arch built with ⊤ irons, as shown in Fig. 4; the pressure was continuously applied up to about 27 500 lbs., at which point the concrete separated from the inside flange of the outside ⊤. In the underside of the arch hair cracks appeared, but there was no separation along the web of the middle ⊤. During

the time of observation, pumping was discontinued, and the pressure dropped to 22 500 lbs. Then it was applied again, when the hair cracks on the top of the crown opened $\frac{1}{16}$ in. at 30 000 lbs. At 32 000 lbs. the outside T went out $\frac{3}{32}$ in., and the top surface of the concrete began to spall. At 35 000 lbs. the cracking on the haunches began. During the progress of this test, one of the 2¼ x ¾-in. tie rods pulled out on the horizontal section just below the bend. The broken end shows rather a poor character of metal failing under direct tension. The curve of loading is shown in Fig. 19. The dotted line in this figure is the curve of deflection. The composition of the arch was as follows :

GENERAL COMPOSITION OF ARCH OF TEST NO. 81.

	Coarse.	Fine.	Coarse.	Fine.	
Sand	4.075	4.075	2.04	2.04	Cement.
Stone	8.150	.....	1.36	1.02	Water.

Test No. 82.—This arch was built with 4-in. I beams. The load was applied at the crown over an area of about 1 sq. ft., and midway between the center and an outside beam. The outside I beam was wedged in place laterally against the upper angle iron used to hold the I beam skewbacks in place, and therefore met the conditions usually occurring in a floor where there would be support on all sides of the points of additional loading. The load was continuously applied for five minutes until it reached 35 000 lbs., at which point there was a separation from the web of the outside beam. At 47 500 lbs., it cracked along under the beams and cracked also on top at the center. At 52 500 lbs. a second crack appeared in the center, and at 55 000 lbs. after 10.5 minutes the machine clamp broke. The section of the clamp at the point of fracture was 3 x ¾ ins., and the failure occurred at the point where it was bent to clasp the skewback beam. The cracks closed, leaving a permanent deflection of 0.22 in. In this test, the tie rods were moved together immediately under the portion of the arch attacked, so as to have as much strength there as possible. The failure of the arch had occurred, but its strength was still very great. The construction of this arch is shown in Fig. 3, and its composition in detail is given in the following table :

DETAILED COMPOSITION OF ARCH OF TEST NO. 82.

Arch.	Coarse.	Fine.	Compacts.	Arch.	Coarse.	Fine.	Compacts.
a....	0.908	0.908	.....	2 *b*....	1.360	0.908	0.940
	1.135	0.908	1.580		0.908	0.908	1.790
	0.908	1.135	2.460		0.908	0.908	2.450
	1.360	0.568	0.812		1.360	0.568	0.796
	1.360	0.794	1.540		0.908	0.798	1.560
	0.908	0.680	1.690		0.908	0.908	2.330
	1.135	1.360	.....		0.908	1.360	Against skewback and for leveling.
	1.820	.....	.....		2.100	.....	

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 2 *a* AND 2 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	27.90	29.8	13.950	14.89	Cement.
Stone.....	55.75	....	5.575	5.20	Water.

Test No. 83.—This was practically a continuation of Test No. 82, new machine clamps being put in position and the same blocks used at the same position on the arch for the application of the load. The arch was continuously loaded up to 52 000 lbs., requiring 13 minutes; then pumping was discontinued, and observations made, during which time the pressure dropped. Then the loading was continued up to 58 750 lbs., at which point failure was deemed to have occurred, the duration of the test being 17.5 minutes; the arch was still, however, so strong that after the removal of the outside curved I beam, it was difficult work to break it down with a sledge so as to make it fall in, and it would no doubt still have carried a considerable load. At the time of the failure, there were cracks radiating from each corner of the block used for imposing the load to the point of intersection of the enclosing I beams with the skewback beams, showing that the whole material had been crushed down.

Test No. 84.—The load was continuously applied over the crown of the center 4-in. I beam up to 42 500 lbs., when the loading was suspended for a few moments, until a new position of the deflection stake was obtained, and then applied a second time. At 55 000 lbs., the maximum load, the beams separated from the concrete, and the concrete cracked. Continued pumping resulted in no increase of the

pressure, but in an increased deflection and an opening of the cracks, until the deflection bar caught. Fig. 20 shows the curve of loading. The solid line is the curve of loads, and the dotted line is the curve of deflection, the total deflection being 0.585 in. One of the tie rods was a little slack at all times. The composition of the arch is given in the following table :

DETAILED COMPOSITION OF ARCH OF TEST No. 84.

Arch.	Coarse.	Fine.	Compacts.	Arch.	Coarse.	Fine.	Compacts.
1 *a*.....	0.908	0.454	1.000	1 *b*.	1.816	0.908	1.084
	0.908	0.908	0.791		0.908	0.680	0 938
	1.132	1.022	1.178		0.908	1.022	1.000
	0.908	0.680	1.000		1.816	0.908	0.960
	1.360	0.680	0.875		1.360	0.908	0.916
	1.132	0.908	1.042		0.908	0.908	1.178
	0.908	0.908	Key.		0.908	0.908	Key.

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 1 *a* AND 1 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	8.16	4.08	4.08	4.42	Cement.
Stone.....	16.33	.....	2.49	1.815	Water.

Test No. 85.—This was of the same arch, one-half of which was the subject of Tests Nos. 82 and 83. Both the I beams were removed, so as to leave a concrete rib 3 ft. wide ; the tie rods were shifted across underneath the arch, so as to afford as good a support as practicable. The loading was placed eccentrically, 23 ins. from one skewback beam and halfway between the faces of the concrete rib; at 27 500 lbs. the north haunch, which was the one nearest the point of application of the load, cracked. At 31 250 lbs., reached in 4.3 minutes, spalling on the top began, the cracks continued to open, and increased pumping resulted in a decrease in the pressure.

Test No. 86.—This was a test of a combined concrete-iron lintel of the cross-section shown in Fig. 5. The load was steadily applied ; at 13 250 lbs. cracking began in the middle ; at 25 000 lbs. it spalled above the angle on one side near the center, and a well defined crack appeared on top, the cement sliding out at the ends past the angles ; at 32 500 lbs. the bolts which ran from angle to angle, and were imbedded in the concrete, were sheared off between the concrete and the angles, and the material spalled along the angles on both sides.

Test No. 87.—This was of a 4-in. I beam and concrete arch like that shown in Fig. 3, the load being applied eccentrically 1 ft. 7.5 ins. from one skewback, over the central arch beam. The arch was boarded over, and the space over the haunches filled with a leveling of cinders. The pressure was applied continuously until 30 000 lbs. was reached, at which point a stick lodged in the valve from the oil that was being used. Then the pressure was permitted to drop to zero, the valve cleaned, and the pressure reapplied continuously for 13.7 minutes up to 70 000 lbs., at which point the clamps at the north end broke at the bottom, tearing clean off. These clamps were $3 \times 1\frac{1}{8}$ ins. at the point of fracture, and broke where the clamp was bent to encircle the flange of the skewback beam.

Observations Concerning the Melan Arch Tests.—In the last columns of Table No. 1, certain deductions are drawn which are offered as indicating one way of interpreting results that can with propriety probably be interpreted in several different ways. The results of the column headed "per square inch of steel" are taken simply from deduced results in the case of the Tests Nos. 76 and 77, according as one or more of the sections are subjected directly to compression. It is probable that the concrete on the two sides considerably increases the strength of the material by preventing it from buckling as it did when loaded alone. It is also probable that the strength of the concrete is in a measure increased by the I beams; although the figures given are not very conclusive, since in the only case where the concrete was attacked alone it was attacked by an eccentric load, bringing into account its tensile strength, while in all of the other cases, save one, it was permitted to act almost exclusively in compression.

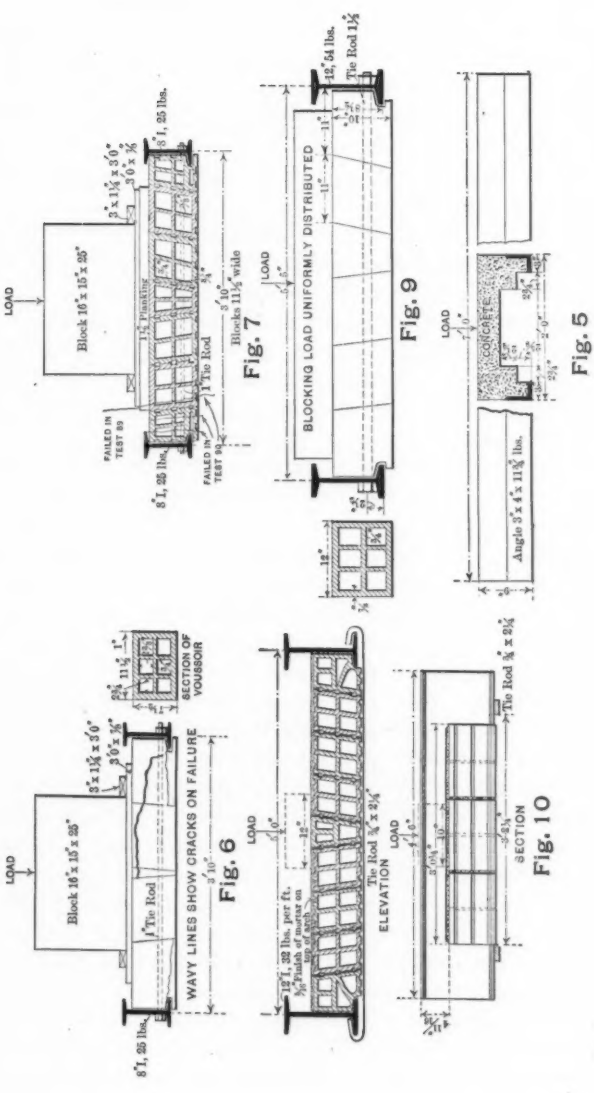
Tile Arch Tests.—These were tests of a series of arches, some of which were built at the works of the Staten Island Terra Cotta Lumber Company, but the majority by the Raritan Hollow and Porous Brick Company, at various times during the winter of 1894–95 and the summer of 1895.

Test No. 88.—This was made on a hollow tile end-construction arch $7\frac{1}{2}$ ins. deep and of an effective depth of about 6 ins., made of hard tile and laid up without any mortar, one block wide. This arch is shown in Fig. 6. The clay blocks were fitted in place over a temporary center and then the key block put in and wedged as well as it could be, but owing to irregularities of the material this could not be very satis-

TABLE No. 1.—MELAN ARCH DATA.

Number.	Span.	Rise	Width.	Areas.		Loads.			Deflection.	Thickness.	Material.	Areas in Compression.		Tie rods. Tension.	Ultimate shear.	Horizontal Thrust.		
				Total.	Loaded.	Total.	Per square foot of area.	Per square foot, loaded.				Steel.	Concrete.			Total.	Per square inch of material.	
																	Steel.	Concrete.
	Ft.	Ins.	Ft.	Sq. ft.	Sq. ft.	Lbs.	Lbs.	Lbs.	Ins.	Ins.		Sq. Ins.	Sq. Ins.	Lbs.	Lbs.	Lbs.	Lbs.	
76....	6	7.00	0.25	.....	.....	14 000	.....	.....	1.62	3	Steel T	1.95	.....	84 250	53 000	36 000	18 450	.....
77....	6	6.66	0.20	.....	.....	14 000	.....	.....	0.87	4	" I	2.05	.....	84 250	53 000	37 800	18 450	.....
78 ..	6	6.97	6.00	36	3.16	4 150	115	131	0	4	Steel, concrete.	6.15	212	168 500	106 000	5 360	.....	.....
79....	6	6 97	3.00	18	3.24	5 320	296	1 642	0	4	"	4.10	106	84 250	53 000	13 740	.....	.....
80....	6	6.97	6.00	36	9.90	12 945	360	1 310	0	4	"	6.15	212	168 500	106 000	33 400	.....	.....
81....	6	8.53	4.00	24	1.00	40 000	1 667	40 000	0.43	3	"	1.95	70.5	168 500	106 000	80 450	18 450	880
82....	6	7.28	3.00	18	1.25	55 000	3 055	44 000	0.47	4	"	4.10	106	168 500	53 000	135 800	36 900	933
83....	6	7.28	3.00	18	1.25	58 750	3 260	47 000	0.59	4	"	4.10	106	168 500	53 000	145 000	36 900	1 019
84....	7	7.52	6.00	36	1.00	55 000	1 528	55 000	0.58	4	"	2.05	106	168 500	106 000	153 800	18 450	1 277
85....	6	7.28	3.00	18	1.50	31 250	1 735	20 850	0.49	4	"	.....	106	84 250	53 000	87 900	.....	839
86....	.....	.....	.....	.....	.....	32 500	.....	.....	0.52	.....	"	.....	.....	...	.....	.....	.....	.....
87....	6	7.28	6.00	36	1.00	70 000	1 945	70 000	0.10	4	"	2.05	106	168 500	106 000	180 800	18 450	1 530

NOTE.—In the calculations here made, the ultimate tensile strength of the tie rods is assumed to be 50 000 lbs. per square inch. The ultimate shearing strength of the rivets in single shear is taken to be 30 000 lbs. per square inch.

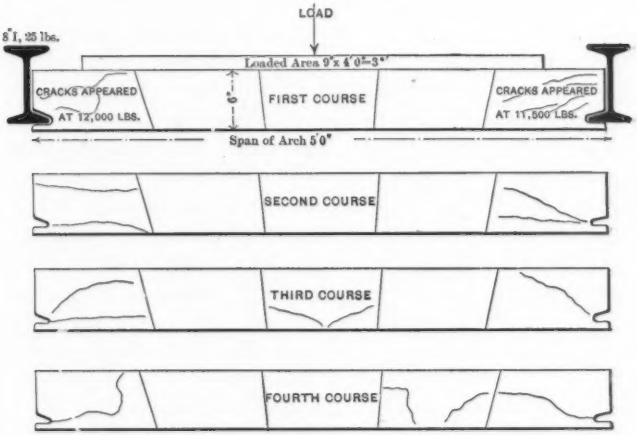

LOAD
Block 16" x 15" x 35"
8" I, 25 lbs.
3" x 11½" x 5'0"
3'0" x ⅜"
1½" Planking
1" Tie Rod
FAILED IN TEST 89
FAILED IN TEST 90
Blocks 11½" wide
Fig. 7
BLOCKING LOAD UNIFORMLY DISTRIBUTED
12" 64 lbs.
Tie Rod 1½"
Fig. 9
CONCRETE
Angle 3" x 4" x 11⅝ lbs.
Fig. 5
WAVY LINES SHOW CRACKS ON FAILURE
SECTION OF VOUSSOIR
Fig. 6
12" I, 32 lbs. per ft.
½" Finish of mortar on top of Arch
Tie Rod ¾" x 2¼"
ELEVATION
SECTION
Fig. 10

factorily accomplished. The pressure was continuously applied, and at 5 000 lbs., the key at the upper portion split in a plane parallel with the tie rod. At 6 250 lbs. the skewback on the right-hand side cracked, and a spall flew off the middle voussoir on the right-hand side of the key. At 9 000 lbs. the arch failed along the wavy lines shown. The application of the load was by means of planking leveled up with sawdust over the top of the arch, so as to effect a practically uniform distribution of the loading.

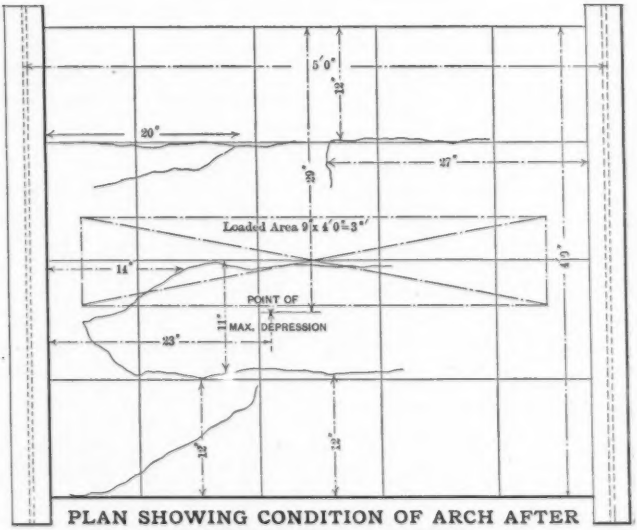
Test No. 89.—This was of a side-construction or old style arch laid up dry like No. 88, and of hard tile. Its construction is shown in Fig. 7. The pressure was applied as shown, there being a slight leveling of sawdust under the planking. There was no perceptible effect until the load reached the maximum of 2 500 lbs., when the arch suddenly collapsed by shearing off the voussoir next the skewback at the point shown. The arch was composed of one course of voussoirs laid up dry; the load was practically uniformly distributed.

Test No. 90.—This was similar in all respects to No. 89, except that in this case, not only the voussoir next the skewback, but the skewback itself, failed by shearing.

Test No. 91.—This was of a hard tile end-construction arch which had been built for some time, and had for several months carried a pile of brick weighing 12 000 lbs. uniformly distributed over it. A little sand was sprinkled over the top, and the load was applied over an area of 9 x 48 ins., or 3 sq. ft., in the center. The point of application of the load was over one of the central longitudinal divisions, and as each one of the blocks composing the arch was about 1 sq. ft. in area, two rows of blocks received the loading. As the load was applied the arch continually deflected until the maximum of 13 750 lbs. was reached, after which continued pumping resulted in a decrease in the amount of the load. The cracking of the various courses is shown in Fig. 8, which does not, however, show how markedly the top horizontal section of the arches was sheared off, showing that it acted truly in compression. There was a certain amount of elongation of the tie rods, but they remained intact. This arch had an effective depth of 4 ins.; the balance being below the skewback and acting simply as fire-proofing for the upper section. In a plane perpendicular to the line of pressure, the section through the arch would show a top horizontal and a bottom horizontal member, a middle horizontal member, and in each



ELEVATIONS SHOWING CRACKS THAT APPEARED UNDER MAXIMUM LOADING.



PLAN SHOWING CONDITION OF ARCH AFTER PRESSURE WAS REMOVED.

FIG. 8.

block three or four vertical members. The arch was laid up in a mixture of one part Portland cement and two parts sand, and over the top of it was a leveling of the same mortar about ¼ in. thick; when taken off, chips of the blocks adhered to this, showing that the tile section was reinforced by about this same amount. Fig. 21 shows the load curve.

Test No. 92.—This was of an end-construction arch, nominally 9 ins. deep, and of 5 ft. 5 ins. span, 4 ft. 2 ins. wide, with an effective depth of about 7¾ ins. The tie rod was so placed as to be 2½ ins. above the bottom, thus reducing the distance from the center of the tie rod to the top of the arch to 6 ins. The section of the voussoirs, which were of hard tile, is shown in Fig. 9. Blocking was so placed over the arch as to distribute the load uniformly over it, and then the pressure was gradually applied until the machine clamp broke at 57 500 lbs. At 40 000 lbs. a slight crack appeared in the bottom of one of the skewbacks running out horizontally from the bottom. At 40 500 lbs. the bottom of the joint of the key block on the left-hand side opened ⅛ in., and on the right-hand side it opened $\frac{1}{16}$ in. At 42 500 lbs. the top began to spall a very little at the center. After the clamp broke the cracks on the arch all closed, and the only evidence that it had been subjected to load was a spall the size of a thumb-nail at the top corner of one of the blocks, and a small crack at the skew in one of the others. This arch was laid in Portland cement mortar, and had set 24 hours when tested, but the temperature had been so low that the mortar had frozen.

Tests of Arches Built for the Purpose.—Table No. 2 gives the weight and composition of these arches. They were laid up in a strong Portland cement and had about three months in which to set.

TABLE No. 2.—Special Arches for Testing.

Test Number.	93	94	95	96	97	98	99	100	101
Depth of block, inches	8	8	8	8	10	10	12	8	8
Weight of block alone, per square foot, pounds	31.3	32.1	36.1	32.5	41.6	40.3	45.0	34.8	32.6
Weight of arch, per square foot, pounds	42.6	42.3	47.4	44	57.5	53.8	57.4	47.0	42.7
Weight of blocks, total, pounds	469.25	481.5	531.5	493.25	624	620.75	769.5	550	514
Weight of sand, pounds	86	78	83	88	116	102	110	96	82
Weight of cement, pounds	68	60	66	70	97	84	82	77	63
Weight of water, pounds	32	30	32	34	49	42	44	38	32
Total weight, pounds	655.25	649.5	712.5	685.25	886	848.75	1005.5	761	691
Total weight, less half water, pounds	639.25	634.5	696.5	668.25	861.5	827.75	983.5	742	675

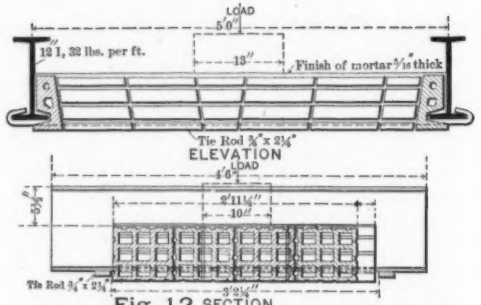


Fig. 12 SECTION

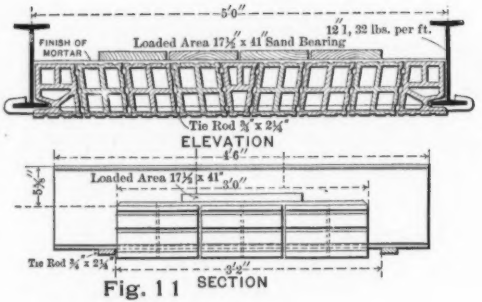


Fig. 11 SECTION

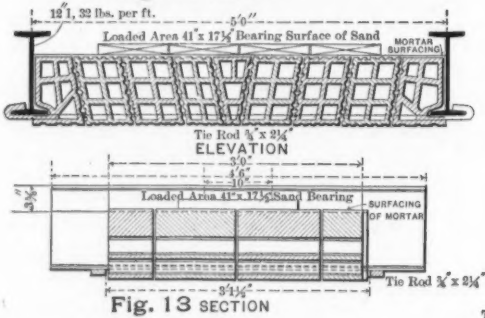


Fig. 13 SECTION

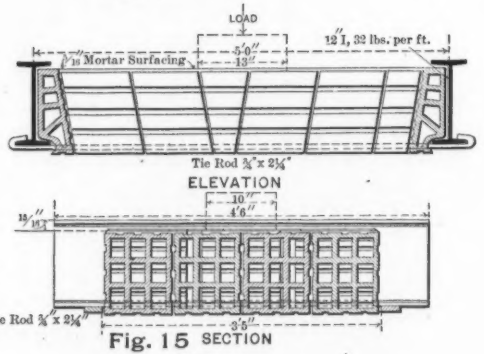


Fig. 15 SECTION

Test No. 93.—This was a hard tile side-construction arch with 8-in. blocks, the effective height of the arch being 7 ins. It was laid in Portland cement with a leveling of about $\frac{5}{16}$ in. of cement on top; the load was applied near the center on an area of 10 x 12 ins. (see Fig. 10). The tying was done by hook rods placed as shown. At 5 250 lbs. the first crack appeared, and the key opened about $\frac{1}{16}$ in., at 8 750 lbs. the crack opened wider, and cracks appeared at the left skewback, opening wide; at 10 000 lbs. the arch collapsed, and cracks appeared at the left skewback; there was a crushing in of the voussoir along its entire length under the right-hand edge of the block on which the load was applied.

Test No. 94.—This was of a side-construction hard tile arch, shown in Fig. 11. The loaded area was $7\frac{1}{2}$ x 41 ins. The load was continuously applied; at 5 000 lbs. there was a slight opening of the skewback; at 5 700 lbs. there was a failure of the east skewback by shearing. Fig. 1, Plate IX, shows the nature of the failure.

Test No. 95.—This was of an arch of hard tile end construction, with porous terra cotta solid skewbacks; the effective height of the arch was $5\frac{3}{4}$ ins., although the blocks were nominally 8 ins. There was $\frac{5}{16}$ in. finish of mortar on the top, and the load was applied on an area of 10 x 13 ins., as shown in Fig. 12. At 5 000 lbs. there was a slight hair crack at the junction of the skewback block with the skewback I beam, and a slight crack at the right-hand side and bottom of the key block. There also appeared a slight hair crack in the mortar cover of the top of the arch. At 6 250 lbs. the crack on the right-hand side of the bottom key opened $\frac{3}{32}$ in., and the top surfacing began to crack. At 7 500 lbs. it cracked in the middle and rose, and the crack in the key opened $\frac{1}{8}$ in. At 8 750 lbs. the top cover bulged further, and the arch opened $\frac{3}{8}$ in. at the bottom of the key. At 10 000 lbs. the top surfacing had bulged 1 in., and the plaster began dropping out (see Fig. 3, Plate IX). At the left-hand end of the arch, the joint between the skewback block and the end construction block opened, and at 11 250 lbs., the maximum load, the arch practically failed, the top cover bulging and cracks radiating out in it from each corner of the block on which the pressure was applied, at an angle of about 45° with lines drawn perpendicular and parallel to the skewback beams. The pumping was renewed, when the bottom of the skew dropped off at the left-hand end, being sheared

PLATE IX.
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HILL ON FIRE-PROOF FLOORING MATERIAL.

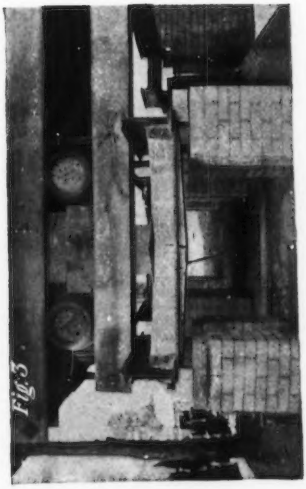
Fig. 3

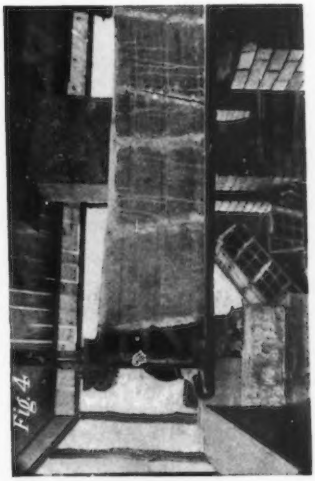
Fig. 4

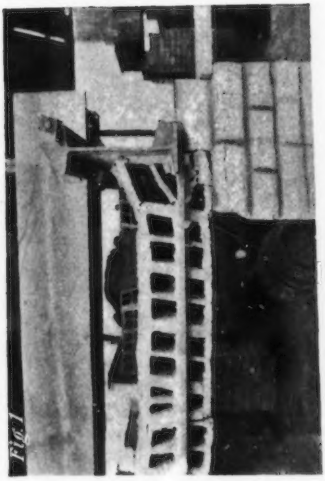
Fig. 1

Fig. 2

through. Then the two east skews, or the parts of the skews which protected the bottom flange of the I, dropped off, and a spall appeared in the bottom of the first voussoir away from the skew, at the left-hand end. At the right-hand end the flange protection dropped off, and cracks appeared running to the right and left from the upper half of the lower hole in the skewback block. Fig. 22 shows the curve of loads for this test and for No. 96.

Test No. 96.—This was of an end-construction arch similar to Test No. 95, and shown in Fig. 12. At 5 000 lbs. cracks appeared at the bottom on the right-hand side of the key. At 10 000 lbs. these cracks opened $\frac{1}{8}$ in.; the top surface bulged up. At 12 500 lbs. the cracks at the bottom of the key opened $\frac{1}{4}$ in.; cracks appeared between the skewback block and the skewback beam along the left-hand end, and a crack also appeared along the left-hand end in the skew and the first voussoir just above the bottom fillet of the web of the skewback beam, as shown in Fig. 2, Plate IX. This skew then crushed off over the whole bottom, and the facing plaster dropped off from the top. The maximum load was 13 000 lbs. The key was cracked along the top cover plate, and the skew was a splintered mass.

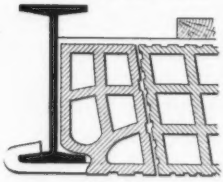

FIG. 14.

Test No. 97.—This was of a side-construction arch of the section shown in Fig. 13, with the load applied over an area of 41 x 17½ ins. The effective depth of the arch was 7½ ins., and there was a leveling of mortar on the top. The key course was quite high in the center so that the load was applied almost directly on it. The course next west of it was also a little high. At 5 000 lbs. a crack opened at the bottom of the key on the left-hand side about $\frac{1}{16}$ in. At 10 000 lbs. failure occurred at the top of the voussoir, the third one from the left-hand skewback, or the second one from the key. All through that course there was a crushing of the top web. The flange protection of the skewback also sheared off.

Test No. 98.—This was of a side-construction arch with a similar loading, namely 17½ x 41 ins. The skewback of this arch is shown in Fig. 14; the remaining details are indicated in Fig. 13. At 5 000 lbs. a crack appeared on the left-hand side of the key at the bottom, and at 12 500 lbs. the top cover began to break up, while failure occurred at 14 500 lbs., the web along the voussoirs next to the skewbacks at

the left-hand end shearing all across and directly down. The effective rise of this arch was 8 ins.

Test No. 99.—This was of an end-construction arch nominally 12 ins. deep, having an effective rise of 10 ins., and with a side-construction skewback of hard tile, shown in Fig. 15. At 3 750 lbs., reached in one minute, a hair crack appeared on the right-hand side of the key, and showed through to the other face. At 7 500 lbs., reached in 2.5 minutes, the skewback began to show failure at both sides. At 10 000 lbs. no change took place. At 11 250 lbs., reached in 4 minutes, the arch again began to fail at the skewbacks, and the crack next to the key at the bottom opened $\frac{1}{16}$ in. At 15 000 lbs., reached in 13.2 minutes, the cracks showed all along the skewback joints on both sides. At 16 400 lbs., reached in 14.3 minutes, the arch failed, the top surface cracking, a slight crack appearing in the top cover of the key, and the skews cracking off just below the diagonally inclined piece on both sides, the bottom cover piece falling off (see Fig. 4, Plate IX).

Test No. 100.—This was of an end-construction arch, similar to that shown in Fig. 12, except the skewbacks, which are shown in Fig. 16, and were of the porous terra cotta solid type; the effective rise was 7 ins., and there was a slight finish of mortar on top. The load was applied on an area of 10 x 13 ins. At 2 500 lbs., after 1 minute loading, a hair crack appeared on the right-hand side of the bottom key. At 5 000 lbs., after 1.5 minutes, a crack appeared on the east skewback by the block moving away from its beam; at 6 500 lbs., after 2 minutes, the crack at the bottom of the key widened to $\frac{1}{16}$ in.; at 7 250 lbs., after 2.3 minutes, the left-hand skewback cracked on the top away from the first voussoir; at 8 000 lbs. the same thing happened with the right-hand skew; at 9 000 lbs., after 3 minutes, the cement cover on top of the key began to rise away from the surface. At 10 000 lbs. this began to crack over the top of the key; at 12 000 lbs. the left-hand skewback began to crack where the lower hole went through it. At 12 500 lbs. the top surface on the key cracked very badly, and the cracks at the skewbacks opened $\frac{1}{4}$ in. each. At 13 500 lbs., after 5.5 minutes, the key at the bottom cracked open $\frac{3}{4}$ in., and the bottom plaster began to fall out, and at failure the point of the left-hand skew sheared off, the key cracked through the middle, near the top section opening about $\frac{1}{2}$

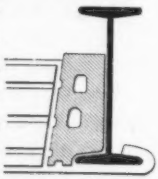

FIG. 16.

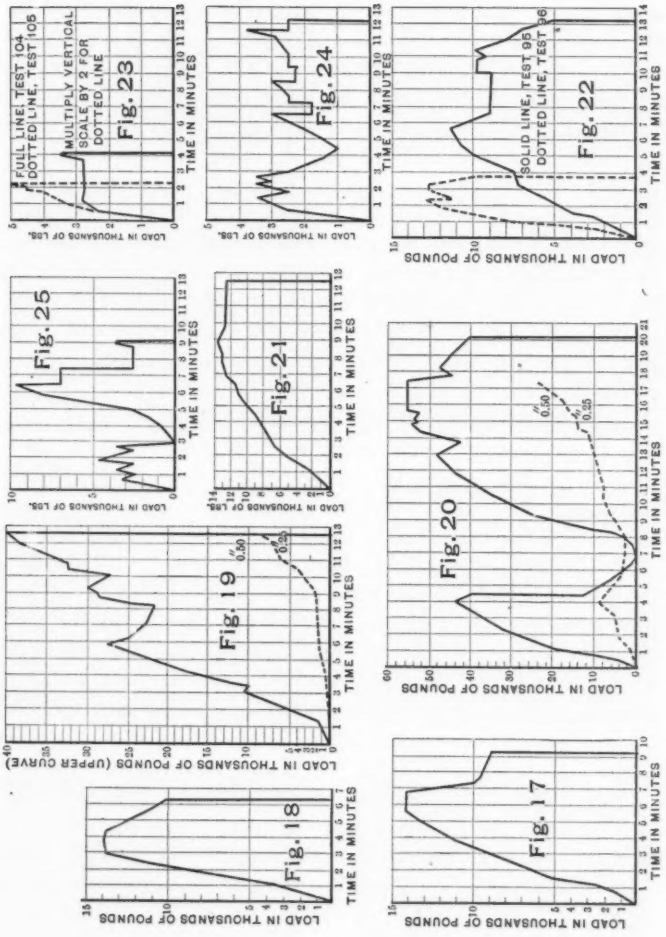
Fig. 17
Fig. 18
Fig. 19
Fig. 20
Fig. 21
Fig. 22
Fig. 23
Fig. 24
Fig. 25
LOAD IN THOUSANDS OF POUNDS
TIME IN MINUTES
LOAD IN THOUSANDS OF POUNDS (UPPER CURVE)
0.50
0.25
SOLID LINE, TEST 95
DOTTED LINE, TEST 96
MULTIPLY VERTICAL SCALE BY 2 FOR DOTTED LINE
FULL LINE, TEST 104
DOTTED LINE, TEST 105

in., and near the bottom section a crack opened that was just perceptible.

Test No. 101.—This arch was similar to the last. At 2 250 lbs. a crack opened on the right-hand side of the bottom key; at 10 000 lbs. this crack opened $\frac{3}{32}$ in.; the top surface of mortar began to spall up. Cracks appeared between the skew and the first voussoir on the left-hand side. At 13 000 lbs. the crack in the bottom of the key opened $\frac{1}{8}$ in., and the cement cover began rising on top in the center. At 15 000 lbs., after four minutes, the crack at the center began to spall, the plaster began falling off at the bottom, and the key began to splinter. When failure occurred the arch was all splintered on the top, and when the top was taken off it was found to have cracked along the line of the arch thrust. The maximum load was 15 750 lbs. The left-hand skew cracked between the two holes, shearing out and dropping off. The first voussoir from the right-hand skew had the bottom plate sheared off entirely, all of the cracks showing the true arch action.

All of the following tests were made with hard tile arches laid up without any mortar. A little damp sand was used to bed the skew-backs of the side-construction arches and to even up the top for the first plank. They were all built between the same pair of beams and usually sagged a little after the center was struck.

Test No. 102.—This was of a hard tile end-construction arch, with a side-construction key, the load being placed as shown in Fig. 1, Plate X, on the top of the key. Immediately on the loading being started, there was a pinching in of the key and cracking at the point where the top of the voussoirs struck it; but the key section of ¾-in. material stood under the vertical load until a maximum of 3 100 lbs. was reached after 2.2 minutes. Data concerning this arch is given in Table No. 3, page 567.

Test No. 103.—This arch was 12-in. end-construction protection-skew hard tile; the key block had the corners chipped out as shown in Fig. 5, Plate X; the left skew was the only one that failed; the right-hand voussoir which had the numbering on cracked on top as shown. The maximum load of 5 000 lbs. was reached in 2.2 minutes.

Test No. 104.—This arch was of the side-construction protection-skew hard tile block type and failed under a distributed load made as shown in Fig. 2, Plate X, which also shows the manner of the failure.

PLATE X.
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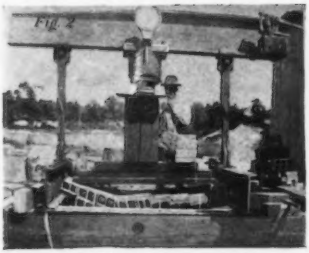
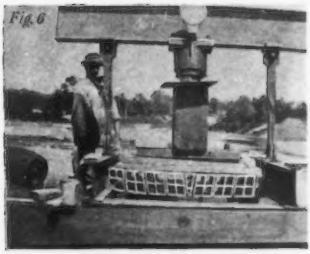
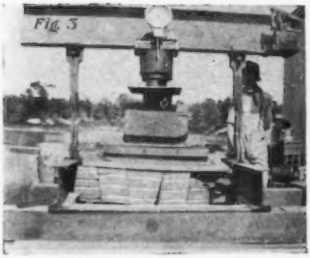
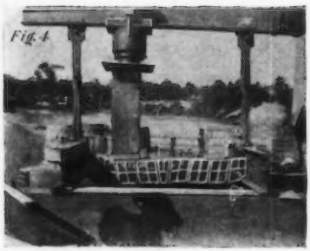
VOL. XXXIV, No. 769.
HILL ON FIRE-PROOF FLOORING MATERIAL.









The only block which failed was the left-hand skew, which sheared off by reason of the thrust of the diagonal piece, not being supported by the flange of the beam. The maximum pressure in this case, 3 500 lbs., was reached in 4 minutes.

Test No. 105.—This arch was built of the same voussoir blocks as Test No. 104, with a new key block and with plain skewbacks, as shown in Fig. No. 6, Plate X. The result was that the arch carried a load of 10 000 lbs., and, when failure occurred, the left-hand skew broke all to pieces. The curve of loading of Tests Nos. 104 and 105 is shown in Fig. 23.

Test No. 106.—This arch was of the 12-in. end-construction hard tile protection-skew type, loaded over 10 minutes, the maximum being reached at 4.5 minutes; the crown deflected about 1¼ ins., due to the crushing in of the material. At the beginning the joint at the bottom of the key was open ¾ in. On the side opposite to that shown in Fig. 3, Plate X, it was practically intact, and, at the time the photograph was taken from which the cut was made, the arch was carrying over 8 000 lbs.

Test No. 107.—This was of a 12-in. end-construction hard tile arch, which began to spall on the top at 2 250 lbs.; the west or left-hand skew split at 2 500 lbs., and the voussoir at the right-hand side of the key split at the top. This arch is shown in Fig. 7, Plate X.

Test No. 108.—This was of a 12-in. end-construction hard tile arch, similar to the preceding, eccentrically loaded, the center of the load being placed 18 ins. from the center of the web of one skewback beam. When the load was placed on this arch the voussoirs at the left-hand side under the point of application of the load slipped past the key block, fracturing and splintering it. Increased pumping resulted in no increase in the pressure. The maximum load was 3 800 lbs. After it was found that no gain in pressure could be made and that the plunger had gone almost the full length of its travel, the load was discontinued and many of the voussoir blocks were found to be intact, except for splintering on the edges. The curve of loads is shown in Fig. 24.

Test No. 109.—This arch was of the 8-in. side-construction hard tile plain skewback type. It was loaded eccentrically with the center of gravity of the load placed 18 ins. to the right from the left skewback beam, and failure occurred by the shearing of the top web at the

left-hand edge of the load block, as is shown in Fig. 4, Plate X. The maximum load of 1 500 lbs. was reached in 1.2 minutes.

Test No. 110.—This arch was similar to No. 109, but centrally loaded. Failure occurred by the collapse of the voussoir next to the key at the left-hand end, or under the edge of the point of application of the load. The balance of the arch remained intact. The maximum load of 2 500 lbs. was reached in 2.7 minutes.

Test No. 111.—This was made on an arch two courses wide, of blocks similar to those of Nos. 109 and 110, with about 1½ ins. of sand covering them; the bearing blocks were 18 ins. wide, 4 ft. 4 ins. long; the lower edge of one skewback broke off at 9 950 lbs. Continued pumping resulted in a decrease in the pressure to 2 500 lbs., and then the second course tumbled in. Fig. 8, Plate X, shows how the left-hand skewback failed by shearing down at the point of junction of the curved rib with the vertical and horizontal portions. This skew was the only part of the course that was injured, which can be seen in the illustration. The curve of loads is shown in Fig. 25.

Test No. 112.—This was similar to No. 111 in all particulars; the loading was on a plank 10 ins. wide and 2 ft. long, centrally applied. At 2 000 lbs. the bottom began to open and there was a little crushing at the crown; at 2 500 lbs., reached in 3.4 minutes, the voussoirs next to the point of application of the load to the right and the one north of it both crushed in, letting down both courses.

Test No. 113.—This test was made on two courses arranged as in Test No. 112, and eccentrically loaded. At 1 250 lbs. one of the voussoirs east of the west skewback cracked through on the edge near the top. At 2 000 lbs. it was cracking off in slivers. When failure occurred, the top voussoir collapsed just to the right of the left skewback. The voussoir immediately under the load also fell in at the point of fracture. The maximum pressure was 2 750 lbs., reached in 2.5 minutes.

It is evident from Table No. 3 that there are not a sufficient number of tests recorded here from which to draw many general conclusions other perhaps than the following:

First.—That the side-construction arch requires a skew in which the inclined member on the line of the arch thrust runs back to a solid support against either the fillet or the flange of the skewback beam.

Second.—The end-construction arch requires good mortar and

TABLE No. 3.—DATA OF CLAY ARCH TESTS.

Test number.	Span. Inches.	Rise. Inches.	Width. Inches.	Total arch area. Square feet.	Loaded arch area. Square feet.	Total load. Pounds.	Load per square foot of arch. Pounds.	Load per square foot loaded. Pounds.	Area tie rod. Square inches.	Horizontal thrust. Pounds.	Shear per foot skew. Pounds.	Depth of skew. Inches.	Blocks. Style.	Blocks. Material.	Blocks. Skew.	Load.	Remarks.
88	46	5	11.5	3.67	2.88	9 000	2 455	3 125	1.57	10 340	4 670	7.5	End	Hard	Protection	Distributed	Failed on arch lines.
89	46	5.5	11.5	3.67	2.88	2 500	682	868	1.57	2 615	1 303	8	Side	"	"	"	Sheared. Laid dry.
90	45	5	11.5	3.62	2.88	1 300	359	452	1.57	1 464	678	8	"	"	"	"	" "
91	60	3.5	48	20.00	3.00	13 750	688	4 590	1.57	29 500	1 720	6	End	"	"	"	Crushed on arch lines. Old arch in Portland cement.
92	65	6	50	22.6	20	57 500	2 545	2 875	3.52	78 000	6 980	10	"	"	Plain	"	Unbroken. Laid in Portland and frozen.
93	60	6	36	15	0.82	10 000	668	12 200	3.38	25 000	1 670	8	Side	"	"	Center	Sheared by edge of load. Portland.
94	60	5	36	15	4.98	5 700	381	1 143	3.38	8 550	950	8	"	"	Protection	Distributed	Sheared by skewback. Portland.
95	60	5	35 2	14.7	0.88	11 250	767	12 800	3.38	33 750	1 890	7.5	End	"	"	Center	Crushed porous terra cotta skew. Portland.
96	60	5	36.5	15.2	0.88	13 000	857	14 800	3.38	39 000	2 135	7.5	"	Porous T. C.	"	"	Crushed porous terra cotta skew. Portland.
97	60	7.5	36	15	4.98	13 500	900	2 710	3.38	13 500	2 250	10	Side	Hard	"	Distributed	Crushed on top by key. Portland.
98	60	8	37	15.4	4.98	14 500	943	2 915	3.38	13 600	2 360	10	"	"	Plain	"	.....
99	60	10	41	17.1	0.88	16 400	960	18 670	3.38	24 600	2 395	12	End	"	"	Center	Side skew. Skew failed. Portland.
100	60	7	38.25	15.8	0.88	14 500	919	16 500	3.38	31 100	2 290	8	"	Porous T. C.	"	"	Porous T. C. Skew. Portland.
101	60	7	38.25	15.8	0.88	15 750	997	17 000	10.14	33 800	2 480	8	"	Hard	"	"	Porous T. C. Skew.
102	60	8.75	10	4.2	0.82	3 100	739	3 780	10.14	5 320	1 490	12	"	"	Protection	"	End skew, side key. No mortar.
103	60	9	10	4.2	0.82	5 000	1 190	6 100	10.14	8 350	3 050	12	"	"	"	"	All end. No mortar.
104	60	5	12	5	1.00	3 500	654	3 500	10.14	10 500	1 750	8	Side	"	"	Distributed	No mortar.
105	60	5.5	12	5	4.0	10 000	2 000	2 500	10.14	13 640	5 000	8	"	"	Plain	"	No mortar.
106	60	9	10	4.16	3.35	15 100	3 630	4 510	10.14	12 560	9 210	12	End	"	Protection	"	Deflection 1½ ins. by crushing in. No mortar.
107	60	9.5	10	4.16	1.00	2 500	601	2 500	10.14	3 950	1 525	12	"	"	"	Center	.....
108	60	9.5	10	4 16	1.00	3 000	723	3 000	10.14	3 980	2 400	12	"	"	"	Eccentric	18 ins. from W. skewback beam. No mortar.
109	60	5.5	12	5.0	1.00	1 500	300	1 500	10.14	3 440	1 750	8	Side	"	Plain	"	18 ins. from W. skewback beam, No mortar.
110	60	5.5	12	5.00	1.00	2 500	500	2 500	10.14	6 850	1 250	8	"	"	"	Center	.....
111	60	5.5	24	10	6.00	9 950	995	1 658	10.14	22 650	2 486	8	"	"	"	Distributed	Two blocks wide. No mortar.
112	60	5.5	24	10	2	2 500	250	1 250	10.14	6 820	625	8	"	"	"	Center	" "
113	60	5.5	24	10	2	2 750	275	1 375	10.14	7 190	1 100	8	"	"	"	Eccentric	Two blocks wide. 18 ins. from W. beam. No mortar.

reasonably good bedding to make the joints which transmit the pressure adequate to their duty, the weak point being the mortar in the end joints.

Third.—When well bedded, the end-construction arch is very much the stronger.

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much